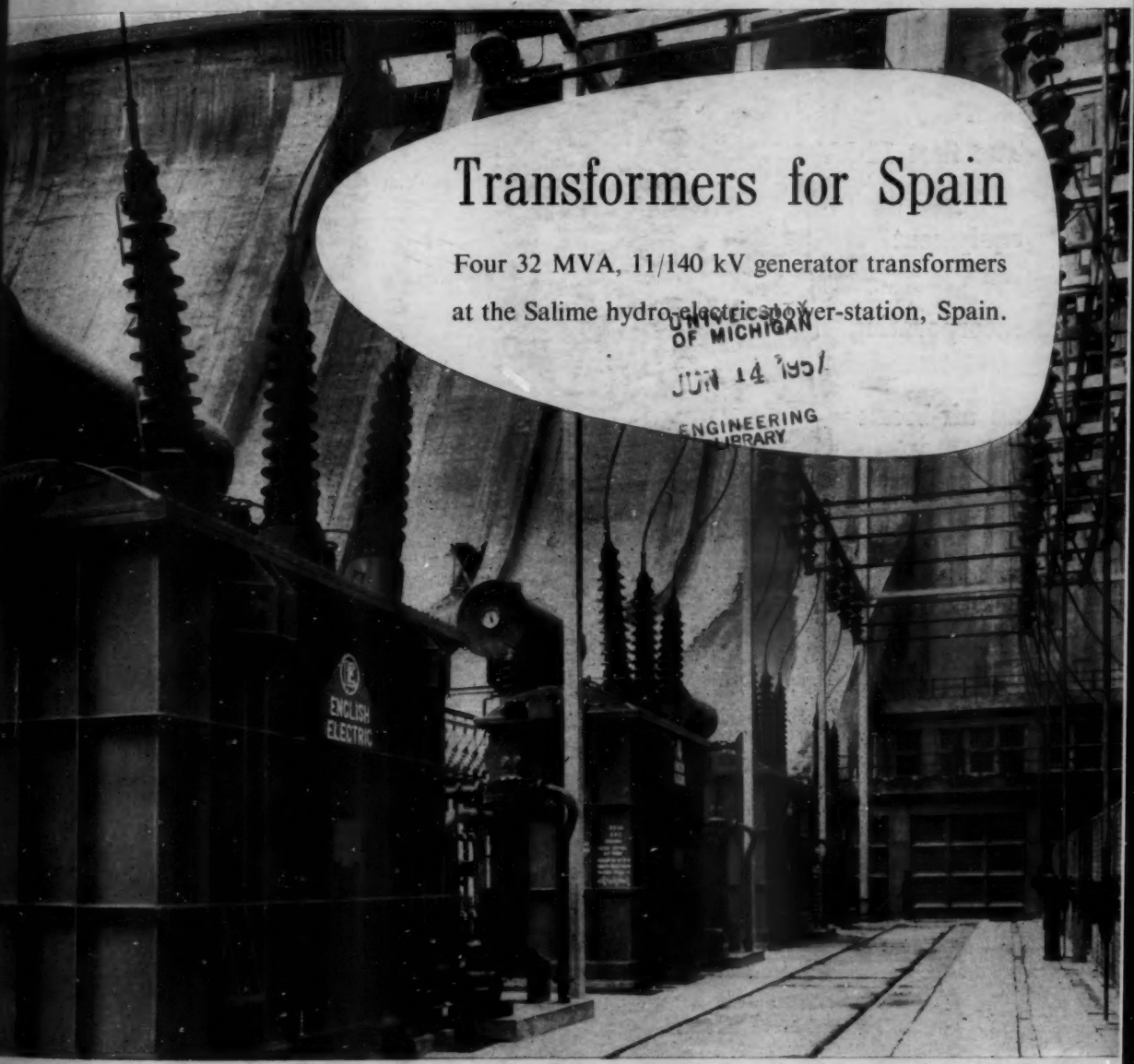
Water Power

INTERNATIONAL DEVELOPMENT OF HYDRO-ELECTRIC POWER

PRICE 2/6 JUNE 1957



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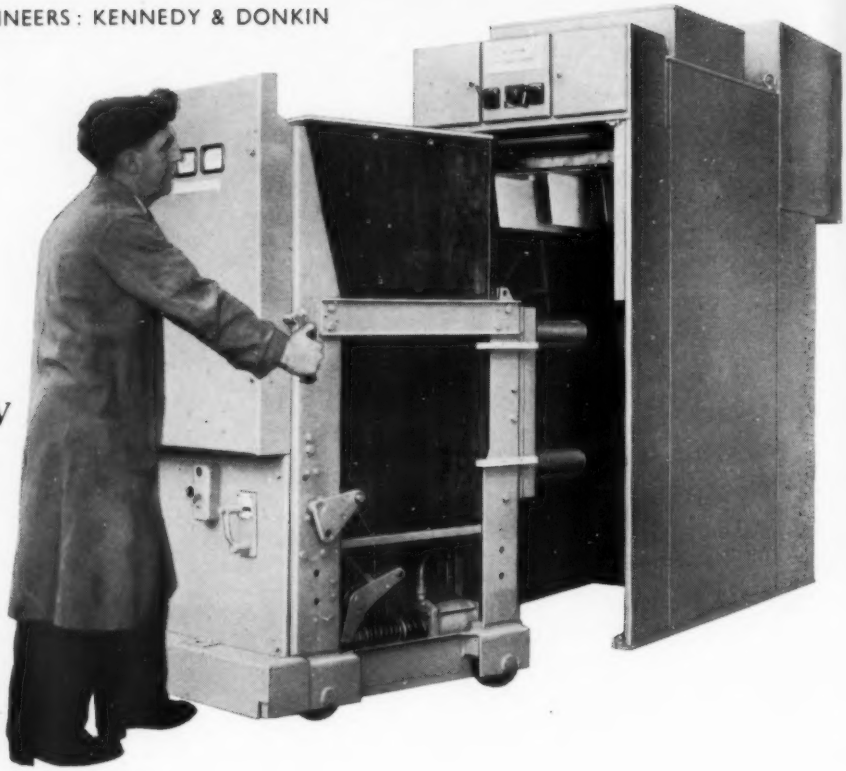

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Volume 9 Number 6

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Editor: H. STOREY, M.C., B.Sc., M.I.M.E.

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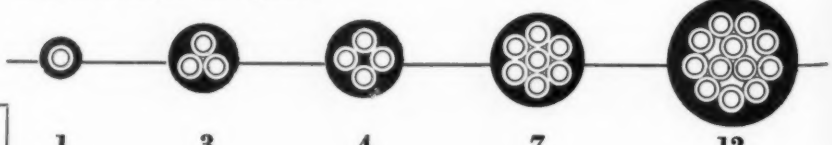

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WATER POWER

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New Tools for Engineers

IN recent months there has been much news of developments in industrial television, and many of these may well have important applications in hydro-electric practice, not only on the constructional side but also on the operation of the final plant. Before touching on some of these developments, however, it is as well to glance at progress in an associated field—that of telecommunications, equipment for which is now available for every sort of industrial use.

Within the last decade there has been a great extension in the use of short-wave radio equipment employed for maintaining communication between different parts of a constructional site. For example, a typical walkie-talkie set, developed in America, now weighs only two pounds and will give reliable telephonic communication over a distance of at least five miles.

For use on restricted areas of perhaps half-a-mile in circumference an even smaller equipment has been devised in England, making use of magnetic induction from a wire loop round the periphery of a building. Such a device could possibly be erected on hydro-electric construction sites, the circumferential wire being cheaply and easily erected on rough poles round the area of work.

The use of portable types of automatic telephone exchange have also been made easier by an adaptation of the type of field telephones used by the Services, providing for easy installation and maintenance. A complete telephone network can nowadays be set up and altered with the utmost rapidity, providing flexible telephone service without the need of an operator.

Loud hailers are nowadays much lighter and more powerful than in the past, and a modern electronic megaphone, equipped with an amplifying device, can easily be carried by a foreman or other supervisor for long periods, with its stentorian voice capable of being heard over half a mile.

Returning now to the television field, cameras have been developed for many special industrial purposes, for example, stereoscopic viewing for remote control of such processes as the handling of dangerous radio-active matter. In the hydro-electric field, the operation of sluice gates under a full head of water can now be observed, either in the normal single-vision fashion, or stereoscopically, using reliable under-water television cameras of small bulk.

Another type of television camera has recently been introduced to enable the inner linings of pipes as small as 8 in. diameter to be examined. The camera, together with a powerful light source, is drawn through the tube, and the operator can halt its progress at any point so as to make a careful examination, taking a photographic record if necessary, of any doubtful sections of the pipe line. There are obvious applications for this type of equipment in hydro-electric constructional and operating techniques.

Cameras are being developed in which the pick-up tubes are sensitive to X-rays, and when these become available there should be many applications in the manufacture of hydro-electric plant.

Television cameras are nowadays made so that the focusing and horizontal and vertical movements can be entirely controlled from remote distances. If such equipment were to be installed at various points on a constructional site, the engineer in charge, from his site office, would be enabled to observe progress in many points at once, and thus be instantly ready to take appropriate action if a bottleneck appeared to be likely.

On the operating side there are a number of applications of television which may well be seen on hydro-electric sites in the near future. For example, a camera could be installed overlooking the trash racks guarding the sluice gates at a remote reservoir, and signs of building up could be detected without having a man available to observe the conditions at the intake point.

The use of suitable under-water television cameras, with very powerful lighting equipment, can enable

the swirl of silt to be observed. The building up of silt masses at the bottom of reservoirs could be continually kept under observation.

Also, on the operating side, is the possibility of using television cameras on upstream sites to give early indication of the piling up of ice.

In unattended hydro-electric stations, the use of a television camera in the control room, with automatic movement of the camera, can act as a permanent check, since by a suitable grouping of meters the operator, perhaps thirty or forty miles away, can observe conditions in the unattended station with a facility which no supervisory equipment can achieve.

It may perhaps be thought that the introduction of further electronic gear of a relatively delicate type into the construction and operation of water power stations has a number of disadvantages which may be felt, by some engineers, to outweigh the advantages. This need not be the case; in these days industrial electronic equipment is designed to have a robustness which equals that of the main plant itself, and moreover the equipment is usually made up of plug-in racks, so making it possible for spare racks to be substituted for faulty equipment with the greatest of ease, and in the shortest possible time.

Norwegian Advisory Committee on Power

THE Norwegian Government has established a Power Advisory Committee, which is to make a survey of the country's power requirements over a period of years and of the quickest and cheapest means of fulfilling them. The Committee will be an advisory body for the Ministry of Industry on questions such as oil and petrol requirements and imports, the building of new power stations and the development of nuclear power. Trygve Lie, former Secretary General of the United Nations and now County Governor of Oslo and Akershus, has been appointed Chairman of the Committee.

Control of Flow by Gates and Valves

AT a meeting of the Hydraulics Division of The Institution of Civil Engineers recently, a paper was presented on this subject by Horace Denton Morgan, M.Sc.(Eng.), of Sir William Halcrow & Partners. In effect, the paper is a review of the various types of gates and valves available, and discusses the relative merits and weaknesses of different designs when applied to appropriate circumstances in practice. The chief value of the paper lies in the author's comments, which are the outcome of his own wide experience in devising means of controlling water flow by the equipment described.

Orders for Chute-des-Passes

DOMINION ENGINEERING CO. LTD. has been awarded the contract to design and build five 11 ft. diameter spherical valves by the Aluminum Company of Canada Limited. These valves will be installed in the Chute-des-Passes power plant which is being built on the Peribonka River, 145 miles from Isle Maligne, Quebec. The rated capacity of this development is one million horsepower. Each of the five valves will control a flow of water equivalent to 200,000 h.p. It is believed that this represents the highest horsepower to be controlled by a single valve, and that each valve will be the largest and heaviest unit of its type ever manufactured. These valves will be manufactured at Dominion's plant in Lachine, Quebec, and designed by Dominion's own engineering and design staff. Similar valves have been designed and built by Dominion Engineering for the Spray development of Calgary Power Ltd. and for the Aluminium Company's Kemano power house.

An order for the five 200,000 h.p. turbines was recently placed with the English Electric Co. of Canada Limited, and it is expected that the station will be in full commission by 1960.

Two of Brazil's Largest Schemes

WORK on two of the largest hydraulic developments ever attempted in Brazil will go into full activity this year: the Três Marias multi-purpose development on the São Francisco and the Furnas power development on the Rio Grande.

Três Marias consists of a very large earth dam located in the upper São Francisco measuring approximately 2,700 m. in length and 64 m in height and having a volume of 15,000,000 cu. m. of earth. The dam will create a lake with a gross storage of 20 billion cu. m. and covering an area of 1,000 sq. km. This ample storage will enable a regulated flow in the upper São Francisco of the order of 700 cu. m. per sec. (present minimum flow is 100 cu. m. per sec. during the dry season), which will greatly benefit the navigation and minimise the periodical floods. It will also permit considerable expansion of downstream plants such as Paulo Afonso. At the toe of the dam a power house will be installed including eight units each rated about 60,000 kW under a design head of 52 m. Only two will probably be installed initially.

The dam will be built by the Federal Government through the São Francisco Valley Commission (CVSF) and the power facilities and transmission lines by the State of Minas Gerais through Centrais Eletricas de Minas Gerais S.A. (CEMIG). Bids for construction of the dam were taken in February and the contract has been awarded to a group of U.S. contractors composed of the following companies: Kaiser Engineers, Utah Construction, Morrison Knudsen and Raymond Concrete Pile. Work is now commencing, and it is planned to finish the dam proper by 1960 and have the first unit on the line early in 1961. The cost of the dam intake, penstocks and other hydraulic structure plus power house with three units is estimated in 5 billion cruzeiros or approximately 100 million dollars.

Furnas is purely a power project situated in a narrow gorge of the Rio Grande, one of the main tributaries of the Paraná. It is situated near the town of Passos and it is the largest hydro-electric project of South America to date. The Rio Grande plan includes the construction of several plants such as Itutinga (50 MW), now in operation, Camargos (35 MW), now in construction, Peixotos (400 MW), now in operation, and several others in the design stage such as São Miguel (60 MW), Funil (120 MW), Estreito (300 MW) and finally, the largest of all, Furnas, situated between Funil and Peixotos.

Furnas consists of a 90 m. high concrete dam resting on quartzite and forming a reservoir of ample

proportions (close to 10 billion cu. m.). The power house will probably have ten units and the final capacity will be between 1,200 MW and 1,000 MW depending on load studies and interconnected operation with existing steam and hydro plants. The total cost is in the neighbourhood of 15 billion cruzeiros or approximately 300 million dollars. To carry out the plan the Brazilian Government organised an independent corporation—Central Eletrica de Furnas S.A. (C.E.FURNAS), whose shares will be subscribed by the Federal Government, by the States of São Paulo and Minas Gerais and by private power utilities such as the São Paulo Light. The purchasing of equipment not yet manufactured in Brazil will probably be financed by the Export-Import Bank or International (World) Bank.

The plant is under final design and work at the site will probably be started in the dry season of 1958 and the first units will probably go on the line by 1962 or 1963.

New Zealand Benmore Scheme

THE Benmore hydro-electric scheme, which will be about a third as big again as Roxburgh, will be started this year. It is planned to be completed by 1965 and will have a capacity of 480,000 kW and is expected to cost rather more than £26 million. These figures were given to the Executive Committee of the New Zealand Electrical Supply Authorities Association by Mr. W. S. Goosman, the Minister in charge of the New Zealand State Hydro-Electric Department.

The President of the Association, Mr. W. S. N. Rennie, said his Committee's proposals for overcoming New Zealand's power problems would soon be handed to the New Zealand Cabinet for consideration. To carry out the power programme envisaged for the next twenty years would cost about £500 million.

Benmore, on the South Island, forms part of a plan to harness the Waitaki river to provide 1·5 million kW of capacity. The full development plan depends on the feasibility of the Cook Strait cable scheme, which will link the two islands with a power cable having an initial capacity of 600,000 kW with provision for an ultimate increase to 1·2 million kW. This capacity, it is considered, would be sufficient to convey all the output of the Waitaki river.

Ffestiniog Pumped Storage

THE English Electric Co. Ltd. have received a contract from the Central Electricity Authority for the four 105,000 h.p. water turbines, governors, storage pumps and valves for the Ffestiniog pumped-storage power station in North Wales. The storage will raise water from a lower reservoir into an upper reservoir when surplus power is available, and this potential will be held in reserve to satisfy peak demands.

In the power station the storage pumps will be installed below the turbines and connected to them by disengageable couplings. The turbines, of 105,000 h.p., will be rigidly coupled to the generator/motors. They will be the largest output water turbines of any type to be installed in the United Kingdom and will operate under a maximum head of 1,050 ft.—the highest head for Francis turbines installed in the country.

Sulzer Bros (London) Limited, in association with Sulzer Bros. Limited, of Switzerland, are the subcontractors for the storage pumps, each of which will absorb a maximum of 105,000 h.p. These units will be the largest output multi-stage vertical shaft storage pumps yet built and will each weigh approximately 292 tons.

All the English Electric work—as well as the larger pump components—will be carried out at Netherton Works (Liverpool), within about eighty miles of the power-station site. The station is due to start operation in August, 1961.

Power Possibilities in British Guiana

THE Government of British Guiana are investigating the possibilities of setting up a public corporation for the supply of electrical power, and if this scheme materialises the assets of the Demerara Electric Co. Ltd. would be taken over and expanded with the aid of the Colonial Development Corporation. Already a survey is being made of the potentialities of the Great Falls, on the Demerara river, with a view to ascertaining whether bulk power could be provided at a sufficiently economic figure for the reduction of bauxite, this being a possibility in which the Demerara Bauxite Company is greatly interested.

New Italian Plants in the Alps

THE hydroelectric potential of the Italian drainage area on the Alps is estimated to be approximately 40,000 million kWh. Nearly 60% has already been developed, so that we may assume that the less-costly plants have already been built. Nevertheless a large amount of work is in progress or prospected for the near future. Among the plants newly built or in an advanced stage of construction, the Plima Lasa plant finished in 1956, the Valgrisanche, Cimego and Somplago plants which are being completed, are particularly noteworthy.

The Plima-Lasa plant, built by the Società Idroelettrica Atesina is a new stage of the Val Venosta (Upper Adige) scheme. It is a high-elevation plant, its main works being situated from between 2,000 and 1,800 m. and its head reaching 983 m. A large dam stores the water of the Rio Plima, a right-bank tributary of the River Adige. The dam is a buttress structure comprising 17 elements. Its maximum height is 83 m., crest length 380 m., volume 310,000 cu. m. The cross sections of the buttresses are isosceles triangles with similar inclinations on the two faces (0·42 and 0·45 respectively). An 11 km. tunnel, entirely lined with concrete, conveys the water to the surge tank, collecting also the water of other tributaries of the River Adige. The penstock has the unusual length of 2,180 m. The inner diameter ranges from 1·70 to 1·45 m. The power station is equipped with two straightflow nozzle Pelton wheels, flow rate 6·5 cu. m. per sec., which are coupled with an alternator of 50,000 kW capacity.

The Valgrisanche plant—a recent addition to the S.I.P. scheme in Val d'Aosta, near the French frontier (Western Alps)—harnesses the water of the Dora di Valgrisanche. A 70 million cu. m. reservoir at an altitude of 1,770 m. is created by an arch-gravity dam which is nearing completion. The dam is a big arch-gravity structure: its maximum height is 132 m., crest

length 394 m., volume 430,000 mm. The head is 1,010 m. In the underground power station two Pelton turbine sets are already installed with a total capacity of 90,000 kW; a third group of 60,000 kW capacity is now being installed. The transformer station is an important interconnection point, from which start the 220 kV lines to Turin, France (through the St. Bernard pass), and Switzerland.

Three plants in series of the Società Idroelettrica Alto Chiese, still under construction, will utilise fully the Alto Chiese river, which rises from the Adamello glacier (Central Alps, maximum elevation 3,544 m.). A total capacity of 340,000 kW is prospected. Two buttress dams will provide regulation reservoirs, at 1,800 m. and 1,220 m. respectively. The first of them, under construction at Malga Bissina, will have a maximum height of 85 m., crest length 580 m., volume 440,000 cu. m.; the second one at Malga Boazzo, just completed, will have a maximum height of 57 m., crest length 440 m. The Malga Boazzo reservoir feeds the Cimego power house, which is the most important stage of the scheme, where the first 110,000 kW set was put into operation in 1956; a second group has now been completed. They are believed to be the largest horizontal units ever built.

Somplago is a large plant of S.A.D.E., still under construction, in the Upper Tagliamento area (Eastern Alps). It comprises a system of tunnels 40 km. long; an interesting arch dam in a seismic area and a large underground power station where five sets are being installed with a total capacity of 300,000 kVA. More detailed information on this plant as well as on other new S.A.D.E. plants in the Dolomites and Eastern Alps will be found in forthcoming issues of this journal, and it is hoped also to publish articles on the Plima-Lasa and Alto Chiese developments.

Power for Burma

THE Balu-Chaung river project in Burma will eventually comprise three hydro-electric stations having a total capacity of 240,000 kW, constituting by far the largest power scheme in the country. It is being carried out by the Kashima Construction Company of Japan, who are operating under a contract from the Government of Burma. It is understood that the general planning and design work was entrusted to the Nippon Koei Company, consulting engineers. Of the three stations projected, the first will have a capacity of 20 MW, the second 168 MW, and the third 48 MW. The scheme is situated in the mountainous region south-east of Loikaw in the State of Kayah and the power will be transmitted at 230 kV to Rangoon, in the south, and at 132 kV to Mandalay, in the north. Most of the preliminary work, including jungle clearance, approach roads and clearing the foundations, has now been completed.

Swedish Contractors for Spanish Scheme

A SWEDISH contracting firm, Widmark & Platzer AB, of Stockholm, have recently signed a contract with Spain's leading private power enterprise, Iberduero S.A., of Bilbao, to act as technical consultants for the subterranean construction work for a power station at Aldeadávila, claimed to be the biggest in Europe. Situated on the Duero river in Northwestern Spain, the power station is rated for an output of 700 MW. The tunnelling work will require the removal of some 600,000 cu. m. (21,000,000 cu. ft) of rock. A "staff group" of three from the company is already in Spain to supervise the work, which is scheduled to take 3—4 years. The Swedish firm will also supervise the machine installations.

Some time ago this Stockholm firm completed a similar job for a 75 MW station in the Philippines. This assignment, for which a number of Swedish technicians were sent out to the Philippines, is reported to have been worth about Kr.40,000,000 (£2,760,000; $8,000,000).

Projects in Basutoland

A DETAILED survey is at present being made of the higher reaches of the Mabbamatso river, Basutoland, where a hydro-electric project is under consideration by the British Government. A preliminary estimate puts the cost of this scheme at £18 million, of which about half would be debitable to the dam and incidental tunnels, and half to the power station which will be equipped to provide an output of 350 million kWh per annum. The scheme is essentially of the multiple-purpose variety and will not only provide a public power supply but will also be of great value in the development of the gold-mining interests in the Orange Free State and in providing irrigation water to a considerable tract of arid but potentially productive land. A second scheme, the Oxbow Lake scheme, involves the construction of a 400 ft. high dam on the river Caledon, and yet a further scheme is projected on the Orange and Semina rivers.

Uganda Buys a Hydro Station

THE first generating station to give a public supply of electricity in Uganda—located on Kiruba Island in Lake Victoria and using wood as fuel—has been shut down, and the Uganda Electricity Board have purchased a 750 kW hydro-electric scheme near Kikagati on the Kagera river. The station will provide electricity for Mbarara and for a mine in Tanganyika. An agreement has been concluded with the mining syndicate who will run their own transmission line from the power station to the mine. The Board will construct a 33 kV line from the station to Mbarara when arrangements for the provision of capital are completed. These arrangements, the supply of material from the United Kingdom and the construction of the line are expected in all to take about two years.

The Kikagati power station was built in 1935 for the nearby Kagera mine, now closed, and stands on the left bank of the Kagera river. Water is taken to the power station from the river, just above a fall along a 600 ft. long concrete-lined channel. One 750 kW generator is installed but there is room in the existing building for a second machine having a capacity of at least 500 kW, and this second machine is likely to be required within the next few years.

The consulting engineers, Sir Alexander Gibb and Partners and Messrs. Kennedy and Donkin, are supervising the work of overhauling the hydro-electric scheme and obtaining the maximum output from it.

Fig. 1. Aerial view of Labrieville

The Bersimis-Lac Cassé Scheme

The development of the first stage of this important Canadian project is described by O. Perryman

PART ONE

IT was in 1952 that the Quebec Hydro-Electric Commission decided to go ahead with the Bersimis-Lac Cassé development to meet the rapidly increasing demand for power to supply the ever-growing industries in the Province of Quebec. Preference was given to this rather remote site over the possible Lachine and Carillon projects, or No. 3 plant at Beauharnois, which are considerably nearer to the load centre, for the following reasons:

It was not practical to start the Lachine development before the St. Lawrence Seaway project had been approved by the Canadian Government in 1952, and furthermore the additional 500,000 h.p. required for distribution by 1956 could not have been developed in that time.

The Carillon project would provide an output of only 300,000 h.p. which would be inadequate, and the Commission decided to reserve this as a peak plant for the future, to supplement the base-load plants of Beauharnois and Lachine, which operate as run-of-river plants.

With regard to Beauharnois No. 3 plant, construction must coincide with the completion of the Beauharnois Canal, and in spite of the rapid progress on this section of the St. Lawrence Seaway, the Canal could not be completed before 1960.

The installed capacity at Bersimis No. 1 will be 1,200,000 h.p. and the Commission having estimated the cost of the outlay at $190 per h.p. delivered in Montreal, decided that this project was the answer to the problem of meeting the acute demand for power by 1956.

The reader will appreciate the many difficulties and the challenge that such an undertaking presented, in many aspects similar to the Kemano project. First, the existing road of the Anglo-Canadian Pulp & Paper

Fig. 2. View of the completed Desroches dam

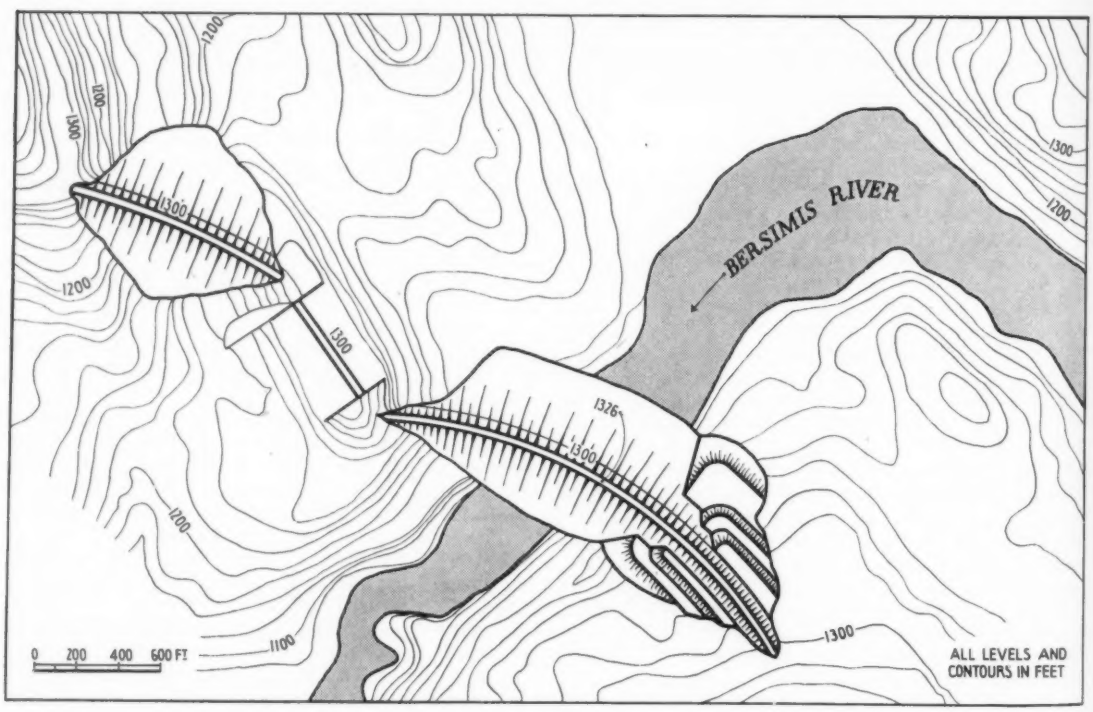


Fig. 3. Desroches dam and spillway

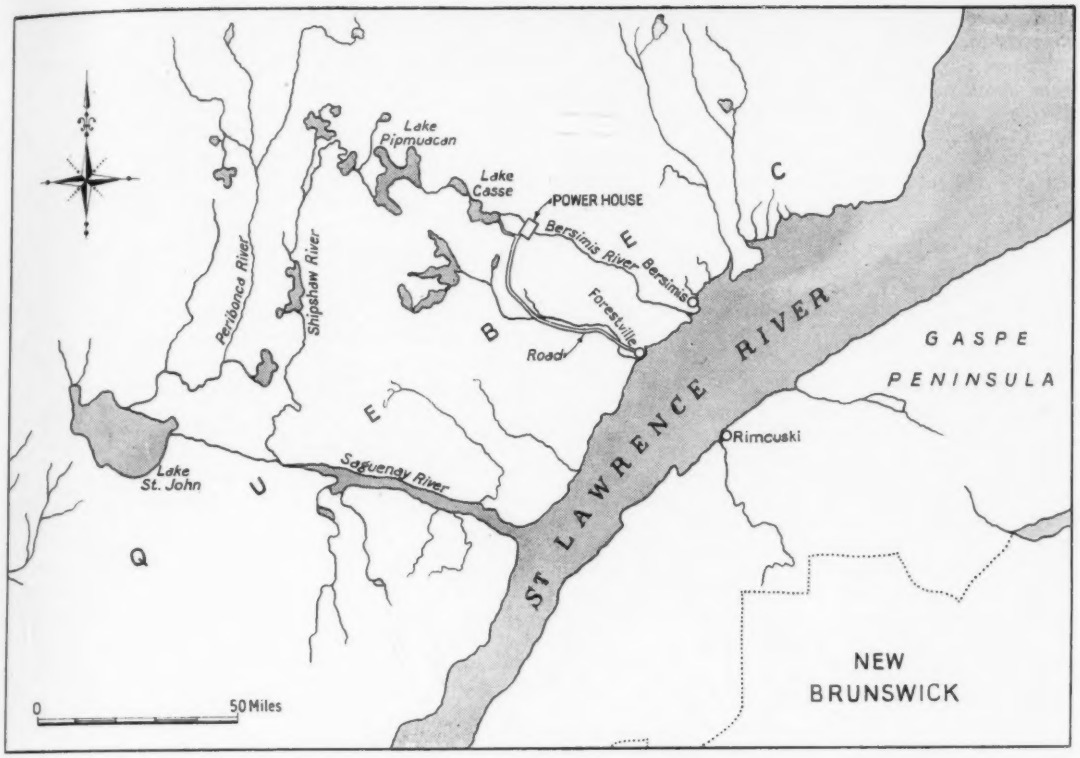


Fig. 4. Sketch map showing power station in relation to St. Lawrence River

Company had to be considerably improved, and its construction extended to Labrieville in order to accommodate the transport of the heavy equipment to the construction sites. A special dock at Forestville had to be built to cater for the ships bringing in machinery and materials. Warehouses, and a special silo for storing cement in bulk were built on the dockside. Then there was the task of establishing accommodation for the labour force at sites which, until then, were a wilderness of spruce-covered hills. Innumerable other obstacles had to be surmounted, but in due time, all problems were resolved.

An outstanding aspect that makes the Bersimis development different from the average hydro-electric project, is that a permanent townsite of far larger proportions than usual was founded at the powerhouse site, and this will eventually accommodate the operating staff. At present it is occupied by the men and their families of the Commission staff on the project, together with the representatives of the many contractors who direct the diverse constructional operations of the whole scheme.

Clearing and levelling operations for the layout of the town of Labrieville, which takes its name from the Bishop of the Diocese of the Gulf of the St. Lawrence, Mgr. A. N. Labrie, was commenced in July 1953 and completed by the spring of 1955. There are now 116 houses, some of seven rooms, in rows of four and six, others being semi-detached or duplexes, and four cottages. These are pleasantly laid out in a semicircle near the bank of the Bersimis with spacious parks between the rows; viewed from a height, the varying colours of the roofs and walls present a gay spectacle in a landscape that would otherwise be rugged and bare from the scourge of forest fires. The architectural design of the Catholic church, with its unusual spire, has been widely appraised for its beauty, and there is also an attractive Protestant church which holds regular services for the non-Catholic community. There is a modern school, staffed by sisters, with eight classrooms each with accommodation for 30 pupils. It caters for both French and English-speaking children up to the standard of eighth grade.

The hotel, completed at the beginning of 1955, is tastefully furnished and boasts the most up-to-date fittings in its 23 rooms. There is a hospital attached to the permanent townsite, and one to the campsite, both of which are fully equipped for major surgical operations. Post office and banking facilities are available, and other amenities on the townsite include a community centre, and a curling rink.

Twenty-four temporary houses, which were built before the permanent ones on a hill north of the townsite, are still in use, and it is intended to move these to the Bersimis No. 2 site later on. This makes the population of the town of Labrieville at present about 710.

The construction camp at Labrieville was completed in December 1953, and provided accommodation and meals for some 1,800 men. There are recreational facilities available on the campsite for the men, such as bowling alleys, a pool room and film shows. The rigorous winters and hilly terrain provide ideal conditions for winter sports, and skiing and ice-hockey are favourite pastimes. In the summer, trout

fishing in the numerous lakes and streams is highly popular, a soft-ball league is organised, and tennis courts are available to all.

At the same time, construction camps were also erected at adits Nos. 1 and 2 to the tunnel, each capable of accommodating 600 men, that at No. 1 also catering for the men working on the intake structure. For the labour force constructing the dam, a camp to accommodate 1,300 men was built at the site, and later, in the spring of 1954, a smaller unit for 175 men operating the cement-batching plant was constructed. At the peak of the constructional operations, the labour force was 5,000 men, while at Labrieville about 70 women were employed as caterers, shop assistants and clerks.

Catchment Area and Hydrology

The first general survey of the Bersimis river as far upstream as Lac Cassé was made between 1945 and 1949 by the Anglo-Canadian Pulp & Paper Mills Ltd., though a preliminary estimate of the undeveloped power of the area had been issued as early as 1922, by the Federal Department of the Interior. The Aluminum Company of Canada obtained some aneroid elevations in 1942, and in 1948 the Federal Government covered the northern portion of the Bersimis drainage by vertical photography. A profile of the river was derived from the aerial reconnaissance made between the Second Falls and Lake Pipmuacan by A. E. Simpson Ltd., in 1949. No direct records of the flow of the Bersimis river had been made, and it was only by a comparative computation of the discharge of the adjacent Outardes river, from the plant rating of the Quebec North Shore Paper Company's power plant at Outardes Falls, where an unbroken record of 18 years' duration was available, that a fair estimate of the Bersimis flow was arrived at.

From all these sources of information, a preliminary study on the hydrology and power of the Bersimis was prepared by the Shawinigan Water & Power Company in 1950, which showed that the possibilities for power development were attractive. However, it was felt that the conclusions reached were necessarily of a tentative nature as the report had been drawn up from data derived by indirect methods, it was decided that a further survey would be necessary. This was undertaken by the Water Resources Department of the Shawinigan Water & Power Company and a detailed report of their results was issued in a Supplementary Report on the Hydrology and Power of the Bersimis in April 1952. It was from this report, and by further investigation that the consulting engineers, Messrs. H. G. Acres & Co., derived much of their design data.

The Bersimis has its course in the Laurentian Plateau about 200 miles north-east of Lake St. John, and flows into the St. Lawrence 235 miles north-east of Quebec City, covering a distance of 260 miles in its course. The total fall in the river is some 1,500 ft., but the most important potential head of 720 ft. occurs in a series of falls and rapids along a stretch of 14 miles downstream from Lac Cassé. In the next 20

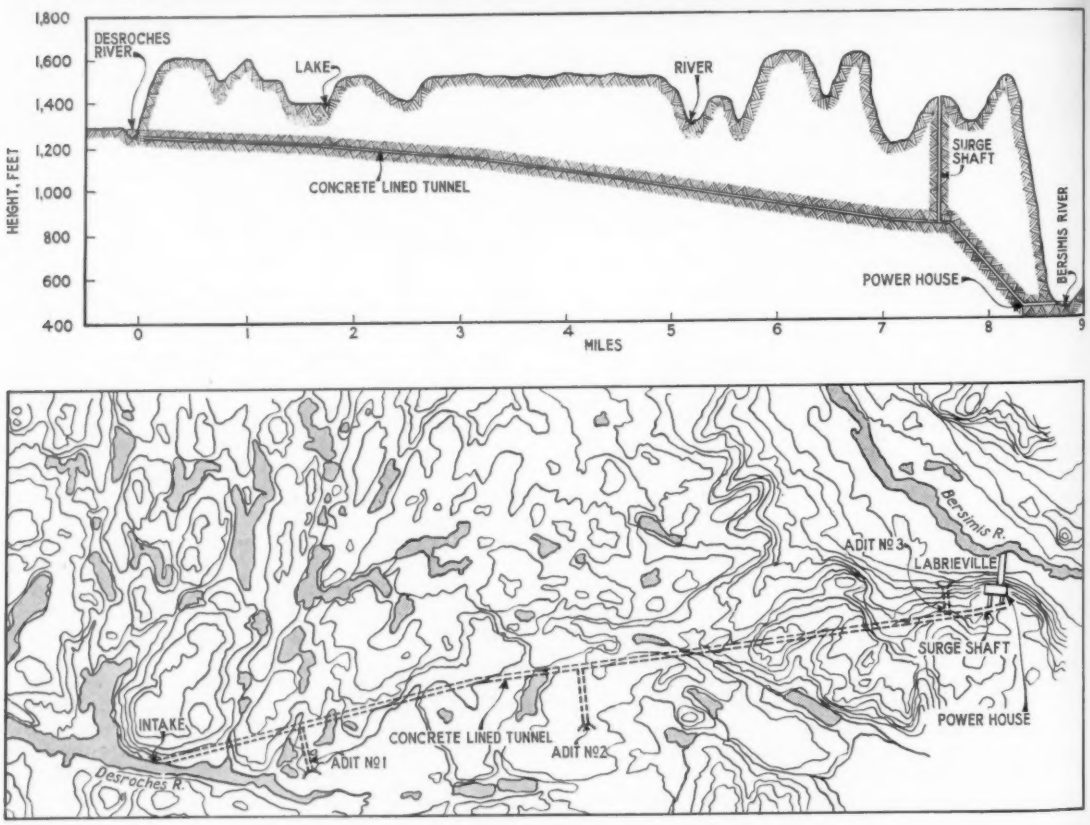


Fig. 5. Sectional elevation and sketch plan showing path of tunnel

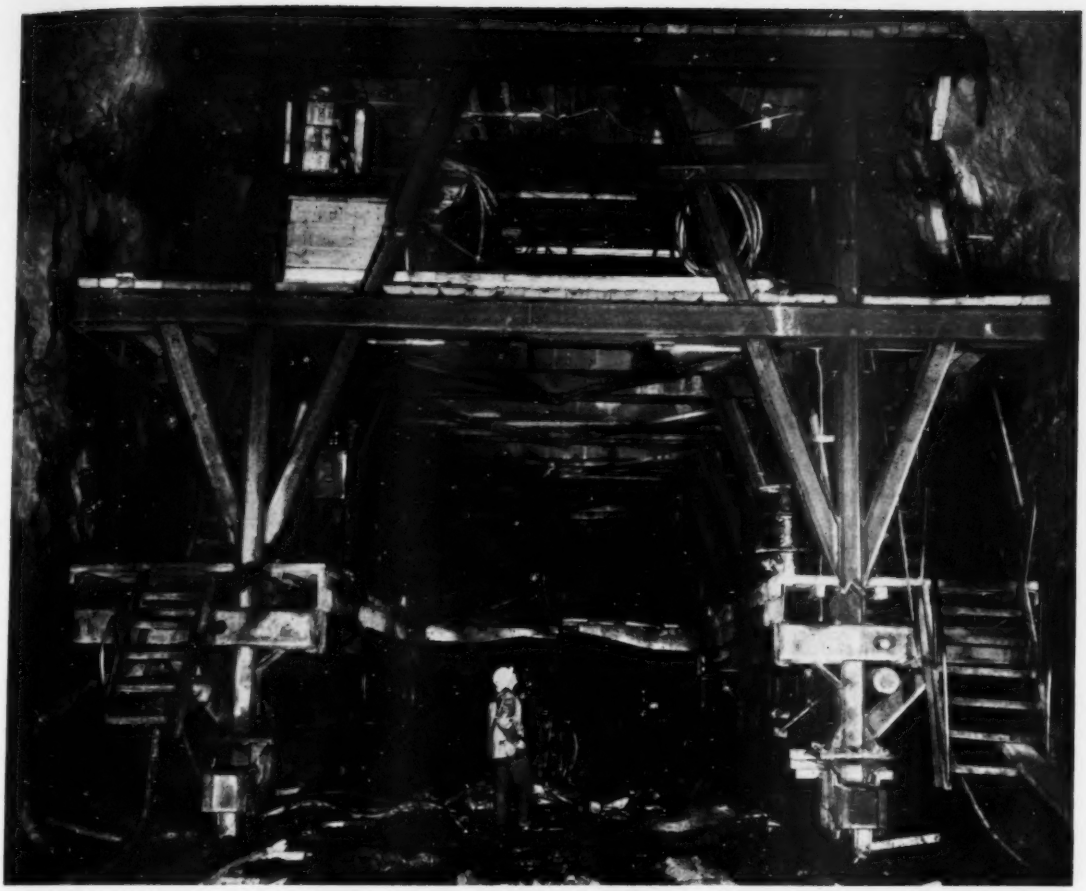


Fig. 6. A view of the drilling jumbo in service in main tunnel No. 3

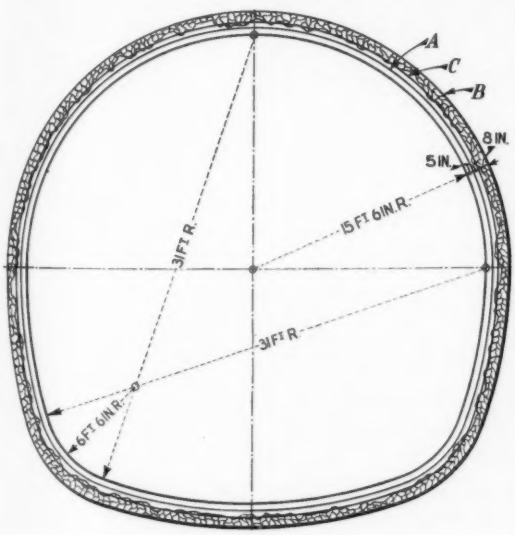


Fig. 7. Cross section of main rock tunnel

miles, there is a drop of 370 ft., which ends just downstream of the First Falls, some 45 miles from the river mouth. It is these two heads which will be utilised at Bersimis No. 1, and Bersimis No. 2 power sites respectively.

The whole catchment area may be sub-divided into the following measured drainage areas:

To the outlet of Lake Pipmaucan	4,500 sq. miles
„ „ „ „ Lac Cassé ...	5,010 „ „
To the Second Falls	5,800 „ „
„ „ First Falls	6,070 „ „
„ „ River mouth	7,110 „ „

Lake Pipmaucan, which will provide the main storage, will have a flooded surface area of 300 sq. miles. It has been estimated that a mean monthly regulated flow of 9,250 cusecs can be maintained with 168 billion* cu. ft. of usable storage capacity.

Storage Dams and Intake

To impound the required storage water level to an elevation of 1,305 ft., it was found necessary not only to build a dam at the Lac Cassé outlet to the Bersimis river but also on the Desroches river. Further, to prevent water from spilling into the Shipshaw and Peribonka watersheds, cut-off dams had to be built at the western extremity of the storage reservoir. This was arranged with Price Brothers whereby two existing dams on their reservoir on Lake Pamouscachiou

* 1 billion = 1,000,000,000

were replaced by dams to raise the lake level by 20 ft. because no suitable foundations were to be found within the Bersimis catchment in that area.

The main dam was built at a distance of 1,200 ft. upstream of the Lac Cassé Falls, and the Deroches dam runs almost in line with it some 2,500 ft. away. In preparation, unsuitable overburden and organic material were removed, and prior to dumping quarry-run rock, a 5 ft. layer of sand was placed on the foundation of the main dam where the overburden was sufficiently impervious and of the required strength to withstand the gravity-type structure. Then on both dam sites a 5 ft. layer of gravel, graded from ¼ to 3 in., was laid.

The dam sites are similar in construction, and are of the rockfill type (Fig. 12), with sloping clay cores. The main body of the dams was formed by dumping quarry-run rock into natural slopes, this being sluiced with monitor jets at 150 lb. per sq. in. while being dumped, in order to reduce the amount of settling in the completed structure. It is interesting to note that the quantity of water used for this operation was four times the volume of rock actually being sluiced.

A filter of three layers, each 7 ft. thick, was placed on the upstream face. The layer in contact with the stone fill is gravel, graded from 3 to 10 in. The intermediate layer is also gravel, graded from ¼ to 3 in., and the outermost layer on which the core rests is composed of sand. This filter prevents the clay core from infiltrating into the rock. A reverse filter consisting of two layers of sand and gravel identical with the last two mentioned layers was placed upstream from the core, and finally a rock-weighting zone was placed having a slope of 1 in 2 for the uppermost 50 ft. and graded to a slope of 1 in 2·5 for the remainder of the dam height. A slight camber upstream was designed to introduce axial compression in the core, shaped in the form of an arch, thus decreasing the possibility of tension stresses.

The thickness of the clay core is 27 ft. at the base of the dam, and 15 ft. at the top. It is made up of glacial till material, and compacted in 6 in. layers to an optimum density of 140 lb. per cu. ft. The foundation below the core was cleaned carefully down to bedrock and grouted to a depth of 40 ft. in stage grouting. On the north side of the Bersimis dam, the core was keyed in a cut-off trench in order to provide an effective seal between the core and the rock, and hence prevent seepage through the rockfill.

At the north end of the main dam, special consideration had to be given to the constructional details, because although the bedrock outcrops at river level on the north bank, the rock surface dips steeply to the north between a deposit of glacial till for more than 100 ft. under the north abutment. Soil investigations, however, proved that the texture was watertight, and of sufficient compressive strength to carry the weight of the dam.

A fixed-crest spillway, 1,000 ft. in length, was provided between the two dams by the process of quarrying surplus rock from the mountain separating them, and this quarried rock was used for filling material.

The main dam is 2,200 ft. long at the crest, 200 ft. high and 900 ft. wide at the base, while the Desroches dam is 1,305 ft. long and 200 ft. high. The following table gives some of the quantities of material used.

Sluiced rock	2,500,000 cu. yards
Rock in weighting zone	1,150,000 „ „
Filter material ...	950,000 „ „
Rolled clay	500,000 „ „
Total ...	5,100,000 „ „

In order to construct the main dam, the river was diverted by means of a tunnel 39 ft. wide, 36 ft. high, and 1,050 ft. long, driven 100 ft. below the river bed. The intake structure for this tunnel is upstream of a rock-and-clay cofferdam across the river at an elevation of 1,175 ft., and it is provided with closure gates 19 ft. wide by 34 ft. high. Eventually this tunnel will be closed off by a concrete plug below the clay core, but at present it provides the water to operate the two units of 5,000 kVA each in the temporary power house. When the water reached the required level, the intake gates were closed, and a regulating valve now admits the minimum flow required for the operation of the two units. A diversion channel 80 ft. wide sup-

Fig. 8. Tailrace looking west from bridge to control room

Fig. 9. Excavation for tailrace open cut in progress

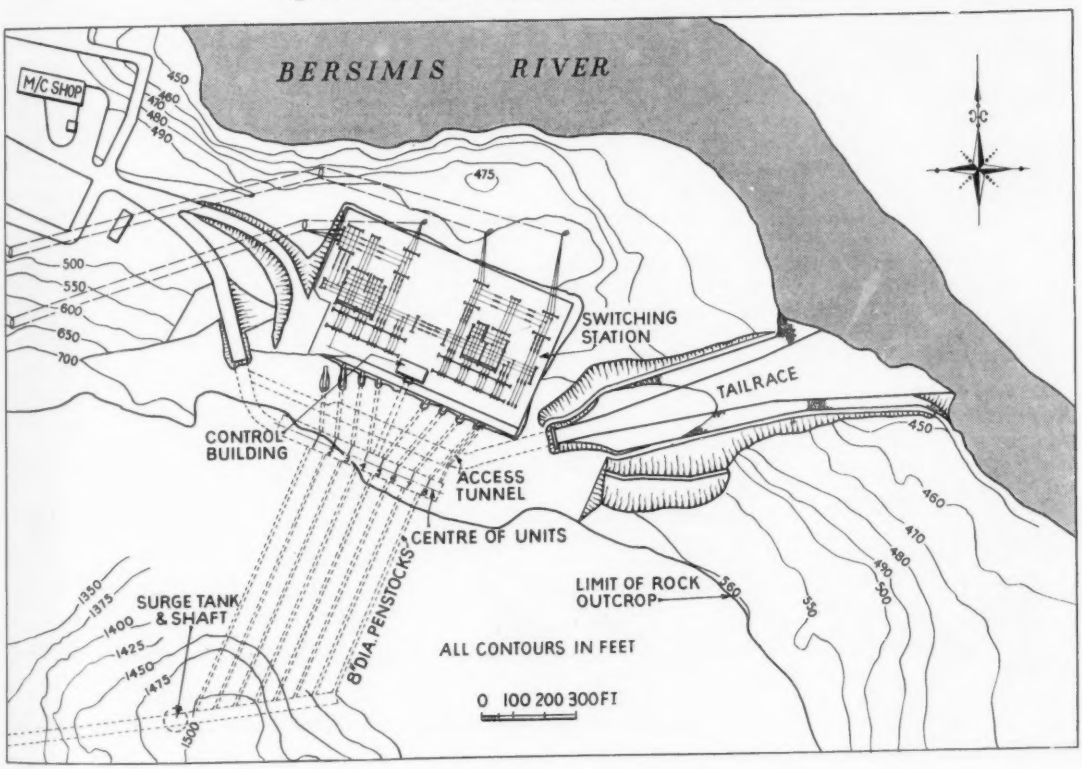


Fig. 10. General plan of power-house area

Fig. 11. Intake portal with trashracks in place

plemented the tunnel, and was located on the south bank of the Bersimis. These two diversions were a precaution against the spring floods from rising above the cofferdam, an event which would have prevented the preparation of the foundations of the dams during the 1953-54 winter and the following spring. As soon as the receding waters permitted the diversion channel was dammed across to allow the main dam to be constructed along the full width of the river.

The diversion of the Desroches river was achieved by means of a 40 ft. square concrete culvert, 1,000 ft. long, built on the rock foundation across the dam. This was quite ample to cope with the flow of the river, which has a watershed area of only 200 sq. miles.

In order to keep up with the time schedule, the diversion tunnel was driven from four headings during the winter of 1953-54. A shaft was sunk at midcourse, offset 30 ft. from the tunnel, and then connected to it by a cross cut. After completion of the tunnel, the cross-cut was sealed by a concrete plug, before the spring flood.

The construction camp at the dam site was completed in the summer of 1953, and the cofferdams, diversion tunnel, and its intake were completed by the spring of 1954. The steel gates for this intake were closed at the beginning of November 1955, to commence impounding ready for operation in November 1956. The rockfill for the two dams was readily available, being blasted from a hill nearby, while glacial till for the impervious core was taken from a pit about four miles from the dam.

The Intake

Owing to the unusual topography of the terrain, the intake structure had to be built some 4½ miles from the Bersimis dam, at a point where the Desroches river bed is at elevation 1,253 ft. The east bank at this point is high and steep, and provides an ideal site for the intake structure, which consists

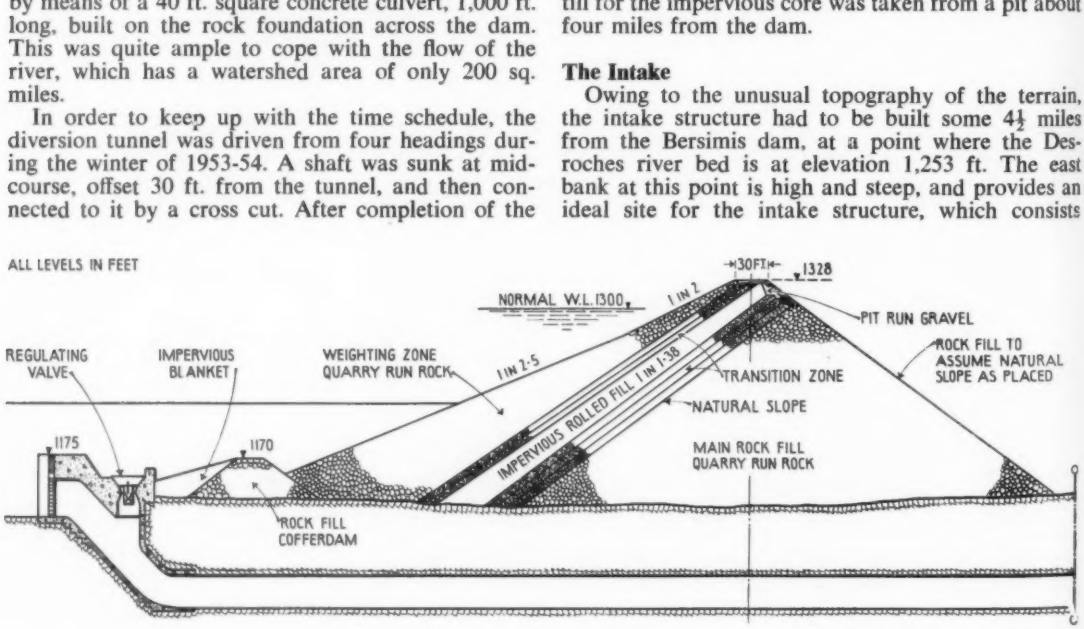


Fig. 12. Cross section of main dam at Lac Cassé, with diversion tunnel.

of eight openings 30 ft. high by 16 ft. wide at the trashracks. Intermediate piers, which are of aerofoil cross section, are so placed as to change the direction of flow through 75° before it enters a chamber adjoining the tunnel. The velocity at peak flow will increase from 4 ft. per sec. at the trashracks to 19 ft. per sec. in the tunnel, corresponding to a loss of head of less than 2 ft. A headgate 33 ft. wide and 32 ft. high, with the sill at elevation 1,235 ft., is situated between the intake chamber and the tunnel entrance. About 90,000 cu. yards of rock have been removed from the bed of the Desroches river, about one mile from the intake, to provide a channel of sufficient cross-section at draw down.

Preparations are being made at the site of the dam for a future emergency channel to divert water down to Bersimis No. 2 in an emergency. At this stage, however, only the stoplogs and foundations for a control gate are being built.

Fig. 13. The surge-tank transition

The Supply Tunnel

A geological survey of the area showed that a tunnel, with sufficient cover, could be driven through almost solid rock without much support being required. The rock in the whole area consists of pre-Cambrian sedimentary gneisses intruded by a graduated series of igneous rock. The sediments contain mainly quartzite, pegmatite and granite. An investigation of the shear zones and lines of fault showed that no great difficulty with poor rock or excess water would be encountered.

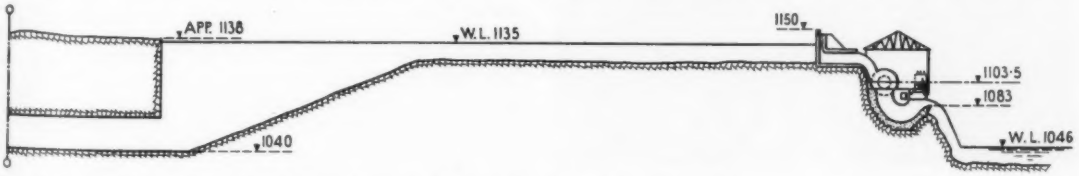
The course decided on for the tunnel deviates very slightly from a straight line over the whole length. The deviation was desirable to secure a better rock cover, and to shorten the length of the adits. The minimum depth in feet of rock cover at any point in the tunnel was specified at not less than 50 per cent. of the maximum water pressure at that point. The designers elected to set a grade limit of 2 per cent. to facilitate the driving of the tunnel.

Owing to the limited time available for driving a tunnel of such large proportions, it was decided to work on six headings. A cost analysis proved that it would be more economical to drive the tunnel from adits instead of shafts. Three adits were driven, one 8,250 ft. from the intake, another 16,000 ft. from the first, and the third near the powerhouse end 14,000 ft. distant from the centre one. These adits measured 25 ft. high and 32 ft. wide, just sufficient for 15 ton Euclids to travel in both directions. Enough space also had to be allowed for water pipes, pump discharge lines, power lines and ventilation ducting.

Once the adits were completed to the tunnel line, full headings in opposite directions were developed, using 18 to 20 power-feed 3·5 in. drifters set on hydraulic booms for positioning. These were mounted at four different levels on a jumbo, a round consisting of from 150 to 190 holes of 2 in. diameter. Tungsten carbide-tipped steels were used, and the holes varied in depth from 12 ft. to 20 ft. Three contractors were engaged on the tunnel excavation, and the blasting pattern and technique differed to some extent; however, an average of 4 lb. of dynamite per cu. yard of rock blasted was consumed in each case. Long-delay blasting caps were used in groups of about 15 intervals, spread over a period of 1·2 seconds.

The muck was loaded into the 15 ton trucks by electric shovels of 1·5 cu. yard design, but fitted with buckets of 2 cu. yards capacity on short boom and dipper stick. Compressed-air plants, each comprising 2,800 cu. ft. per min. vertical compressors, supplied air to the drills at adits 1 and 2. The plant near to the third adit contained three units, and this supplied the drills used in excavating the powerhouse as well.

As had been anticipated, the rock did not require support excepting over a length of 2,000 ft. of faulted ground. Serious water difficulty was experienced at only one place, this being under a lake in the first section. About 2,000 cu. yards of grout had to be



regulating pond, temporary power house with headworks and penstocks

injected into the rock from inside and outside the tunnel to prevent the lake water from leaking into that particular heading.

To pour the concrete lining of the arch of the tunnel, a jumbo was used upon which was set of total of 320 ft. of non-telescopic formwork, the whole moving on a track. The arch was poured in sections varying from 80 ft. to 160 ft. in length, allowing six to nine hours for placing, ten or eleven hours for setting, and in the remainder of the twenty-four hour period, the formwork on the jumbo was moved to the next section. Concrete guns were used in the first two sections and pumpcrete placers in the third. The sequence of operations for concreting the arch made provision for pouring the invert last, after the former had been grouted. In this way, a saving on the cleaning of the invert was effected.

The ventilation system for the tunnel consisted of 38 reversible axial fans driven by 60 h.p. electric motors. The fans were set in series in 48 in. diameter metallic ducts, and the whole system was designed to move the air in the tunnel at a velocity of 50 ft. per minute. To reduce the fumes and smoke from the diesel trucks, scrubbers were attached to their exhaust pipes.

Work on the three adits began in July 1953, and the final break-through in the actual excavating was made eighteen months later. The 18 in. concrete lining of the arch was completed in the autumn of 1955, and the pouring of the invert was completed in July 1956.

The tunnel is 7½ miles long, and its horseshoe-shaped cross-sectional area is equivalent to that of a circle of 31 ft. diameter. The horseshoe cross section was adopted purely to facilitate construction. The excavated diameter is approximately 35 ft., and the average thickness of the lining is 21 in.

The lined tunnel was designed to carry a flow of 13,250 cusecs for a load factor of 70 per cent., using the Manning coefficient of friction $n=0{\cdot}0109$. The loss of head for the tunnel only will be 48·5 ft., and from intake to scrollcase it will be 58 ft., using a coefficient of friction $f=0{\cdot}0087$ (Chezy). The most economical section of the tunnel was set at the limit beyond which any gain in value of power resulting from reducing friction losses by increasing the diameter, would no longer provide a fair return for the additional capital expenditure incurred. An unlined tunnel was considered when it was found that the rock was solid and tough, but further cost investigations showed that a lined tunnel would be more economical than an unlined tunnel for equivalent friction losses.

Fig. 14. Stress relieving a penstock joint

Surge Shaft and Penstocks

Intensive experiments and tests with models of different kinds of surge tanks showed that the restricted-orifice type would be more economical than the differential type. It is located 60 ft. upstream from the first penstock above the axis of the tunnel, and has been hewn out of the solid mountain. The shaft, which is displaced 45 ft. from the tank to allow room for a rock trap at the bottom of the latter, is 27 ft. in diameter and concrete lined. This shaft rises 313 ft. from the tunnel at elevation 822 ft. to the bottom of the tank at elevation 1,135 ft. The tank is unlined and is 365 ft. high and 85 ft. in diameter, emerging at elevation 1,500 ft. at the top of the mountain where a concrete rim supports a roof over it.

To excavate the tank and shaft, a 9 in. hole was bored through from the top; a 12 ft. by 10 ft. pilot shaft was then raised from the tunnel to ground level —a height of 678 ft. Finally the tank and shaft were opened up to their respective diameters by drilling downwards in steps from the top. The muck was passed down through the pilot shaft to a loading pocket in the manifold and thence into 15 ton trucks for disposal.

The eight 12 ft. diameter concrete-lined penstocks branch off from the manifold at the end of the tunnel, sloping downward to the powerhouse at 50° to the horizontal from elevation 792 ft., and at an angle of 60° to the direction of the manifold. The horizontal portion of each penstock is steel lined, and extends for 328 ft. from the elbow to the straight-flow valve in the powerhouse. The diameter of the steel linings decreases from 10 ft. to 9 ft., and finally to 7 ft. 9 in. in the section welded to the straight-flow valve. The steel sections were supplied in 50 ft. lengths, weighing about 50 tons each. These were electrically welded on site, all joints being stress relieved and X-rayed for flaws.

The horizontal sections of the penstock were driven full face from a jumbo mounting four drifters, and mucked with rocker shovels loading into 15 ton trucks. Pilot raises of 6 ft. by 9 ft. were first driven for the inclined sections, then they were opened up to the full diameter of 16 ft. by downhand breast drilling. A 2 ft. concrete lining, and backing in the steel-lined sections, was poured by pumpcrete machines. Pressure grouting in stages up to a maximum of 300 lb. per sq. in. and to a depth of 15 ft. in the rock was carried out to fill in any void, and to provide a watertight curtain around the penstock. The penstocks, valves and scrollcases will be subjected to a pressure of 540 lb. per sq. in., this being the equivalent of a head of 1,240 ft.

(To be continued)

An Experimental Wave Recorder

J. A. Sandover, B.Sc., D.I.C., A.C.G.I., A.M.I.C.E., describes an electrical method of recording rapidly varying depths in a liquid, which he developed in the course of an investigation into surge waves in open channels at the Sanderson Engineering Laboratories, Edinburgh University

ANY experimental investigation involving rapidly varying depths in a liquid requires an accurate and reliable record of these changes in depth with time or distance. Until recently, the standard methods involved the use of a float gauge of some kind, but these suffered from obvious defects, such as inertia and friction, and could not be relied upon unless the depth varied slowly. In recent years, a number of alternative methods have been developed from a consideration of the elementary principles of the theory of electricity. These are divided very broadly into three types: contact gauges, resistance gauges, and capacitance gauges.

With contact gauges a large number of pointers are arranged vertically above one another, and as the fluid makes contact with any one, an electrical circuit is completed. Each addition can then be recorded as an increase in electrical potential.

Resistance gauges comprise two conductors at a suitable distance apart in the liquid. The resistance of the fluid between the gauges then varies directly with the change in depth. These changes in resistance will give rise to proportional changes in either voltage or current in the circuit, and again some method of recording these variations can be employed.

The third method of recording depth changes is the most important, and that most frequently used. If two parallel plate conductors of surface area A are separated by a distance x, and there is an electrical potential between the two plates, then this configuration is termed a capacitor, or condenser, with a capacitance C, and it can be shown that

$$C = \frac{\epsilon A}{x}$$

where ϵ, a proportionality factor, is called the dielectric constant or the permissivity of the media between the conductors, and its value depends upon the media.

There are two methods of measuring depths which employ this information. The first is the "proximity gauge." Here a metal strip is fixed above the surface

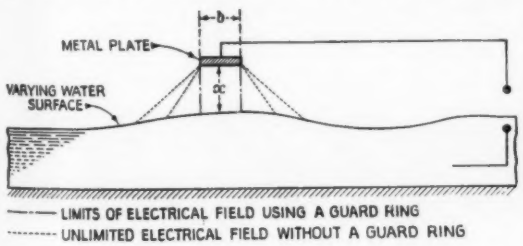


Fig. 1. Proximity capacity-type depth gauge

of the liquid, and as the level between the two rises and falls the capacity of the condenser so formed varies. If this varying capacity is part of an electrical circuit, various methods are available to indicate this change as a varying voltage, which can be recorded.

The disadvantages of this method are shown in Fig. 1. If the breadth b of the conductor and the length of the water surface are small and equal, this method would be most useful, but the water surface extends for some distance in either direction, and fringe effects will be encountered. Thus the capacity will not equal $\frac{\epsilon b}{x_1}$ per unit width of flow, as the areas of the two conductors are no longer equal. The capacity will now be greater since the lines of force in the electric field can follow the path shown by the plain dotted lines, instead of occupying only the area between the chain-dotted lines, as would be the case with conductors of equal area. Effects such as this increase of capacity are termed "fringe effects," and will obviously vary with the distance between the water surface and the metal strip. This variation can be minimised by the use of a "guard ring," which cuts out the fringe effects, but this entails considerable elaboration on the circuitry. The second disadvantage is that with a curved flow profile, unless b is very small, x will vary along b, again introducing non-uniform variation in C. However, unless the curvature of the water surface is very large, this effect is rarely important. The third disadvantage of this method is the limit of the distance x imposed by the configuration of the condenser. Unless the apparatus is very sensitive and yet capable of dealing with a wide range of values of C, x has to be a matter of only a few inches.

This method has been used to good effect in various model tests, with and without a guard ring. If there is no guard ring, then the change in voltage output is not proportional to the change in x. In both cases, however, a calibration process of plotting output voltage to the distance x has to be followed. When calibrating, it is preferable to move the metal plate by fixed amounts rather than to raise and lower the water surface, and then to graph these changes in distance against the changes in output voltage. The great advantage of this method of recording depths is that it offers no resistance to the fluid flow. It should be noted in this type of depth gauge that air is the dielectric medium.

The second type of capacity depth gauge is the "immersed" type. If a piece of glass capillary tubing is filled with mercury and immersed in water, and if the mercury and the water are connected into the same

electrical circuit, this forms a condenser. The capacity of the condenser in this case is

$$C=(x\epsilon)f\left(\frac{r_1}{r_2-r_1}\right)$$

where f indicates some function of r, and r_1 and r_2 are the radius of the wire filament and overall radius of the cable respectively. In this expression x is the immersed length of the tubing, and ϵ is the dielectric constant for glass, i.e., $C=Kx$, where K is some constant. The capacity of the condenser, therefore, varies directly with the change in depth of the liquid.

In general, the two types of capacity gauge have superseded the contact and resistance-type gauges, because the accuracy of the contact gauge is limited to the minimum distance between the point contacts, and apart from other snags, the resistance type of gauge may cause polarisation. Of the capacity gauges the immersion type will now be described in greater detail, since a particular model was developed by the author for a research project.

Investigation for which the Gauge was Designed

A decision was made to investigate the shape of the profiles of surge waves in open channels. These waves are formed when there is a sudden increase in the discharge into the channel, and the shape of the profile varies with the quantity of water added. For small but sudden increases in discharge the surge passes along the channel and the shape of the longitudinal profile can be regarded as sinusoidal to a very close approximation (Fig 2*a*). At larger added discharges the shape of these waves changes, the amplitude increasing and the profile being that of a train of "cnoidal" (cycloidal) waves (Fig. 2*b*). With yet greater additional discharge the first wave breaks (Fig. 2*c*), and finally above a certain added discharge the profile of the body of the surge is almost a straight line (Fig. 2*d*).

A theoretical relationship for the shape of the profile of these undular surges was deduced, and a number of numerical examples were calculated. It was then required to check these results experimentally. A rectangular glass-lined channel, 48 ft. long and of 5 in. by 10 in. cross section was made, with a horizontal bed. Surge waves (Fig. 3) were formed in the channel by a sudden increase in the flow, and it was required to measure the profiles of these waves.

Two methods were considered, and employed to register the surface profiles. The first method involved the use of photography, which gave very good results, but at the best only one wave length could be recorded, due to the difficulties in measurement from the photographs. A fine grid was marked on drawing paper in Indian ink and this was very carefully glued to the side of the channel remote from the camera, but unless the camera was within 6 ft. of this grid it was extremely difficult to measure the profile accurately, and at this distance only one full wave length was included on the photograph. Thus the second method —the immersed capacity depth gauge method—was used, but the two were compared at intervals.

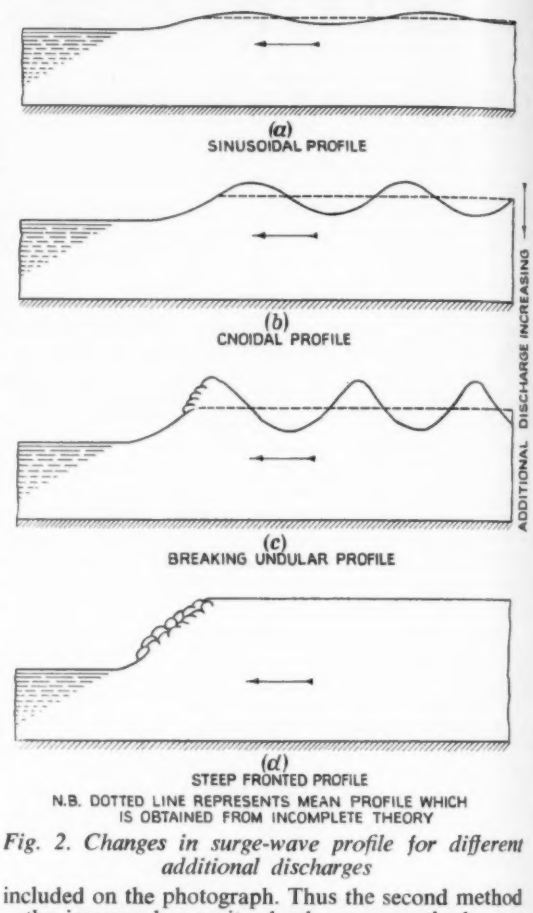


Fig. 2. Changes in surge-wave profile for different additional discharges

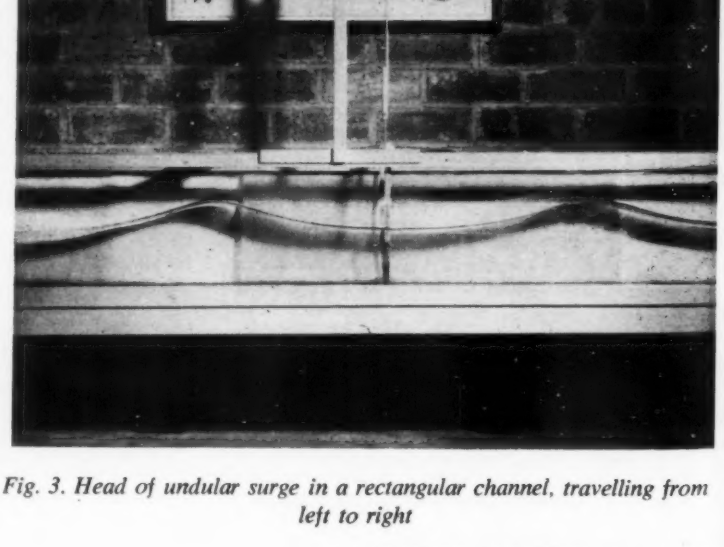


Fig. 3. Head of undular surge in a rectangular channel, travelling from left to right

The importance of the investigation was that usually the designed depths of open channels were obtained by considering the surge as shown in Fig. 2*d*, i.e., using one of the usual theoretical expressions such as that due to Favre or Craya, or by considering it as a moving hydraulic jump. Below the limit of discharge imposed by the breaking of the first wave (Fig. 2*b*), however, the crest height of the first wave of the surge above the initial depth will be at least one third greater than that calculated by the above methods. In many schemes the maximum discharge available is below the quantity required to cause breaking of the first wave, and consequently unless a very liberal allowance of wall height in the channel is made above the maximum depth as calculated from the methods already mentioned, flooding will occur. It is reported in the literature that at the power station at Kembs in France, before modifications were made, the crests of the waves under certain conditions overtopped the sides of the tailrace channel, the depth of the channel having been calculated by the methods mentioned previously.

Immersion Depth Gauge Used in the Experiments

The capacity depth gauge finally used was no longer the capillary tubing described earlier. With even small flows it was found that the combined meniscus effect and the change in level caused by the interruption to the flow gave rise to large errors in the indicated level. The difference was obtained from a comparison with the true level as shown by the photographs. In flows with a mean velocity of approximately 3 ft. per sec., the difference was of the order of half an inch. Various unsuccessful attempts were made to shield the probe, and this particular depth gauge was eventually abandoned.

In the final form of the gauge used, the condenser is formed with either P.V.C. covered cable, or enamelled wire, the P.V.C. or the enamel acting as the dielectric. The use of enamelled wire is preferable, since this is obtainable with a much smaller overall diameter than the most slender commercial P.V.C. covered cable (0·030 in.). This means that the meniscus and "interruption" effect to the flow are much smaller with the enamelled wire as the least diameter available is about 0·010 in. The total errors in the dynamic measurement of depths are of the same order of magnitude with the enamelled wire. Wires of smaller overall diameter are available, but the thickness of the enamel coating has to be the largest available to obtain a sufficient length of wire free from pinholes. If pinholes are present the insulation breaks down, and the condenser no longer exists as such. Another point worthy of note is that certain of the enamels have a capacity that varies with time at a constant depth of immersion, but fortunately the standard makes of this wire with ordinary extra-thick enamel rarely give rise to this trouble.

In the experiments mentioned above, it was decided however to use P.V.C. covered cable owing to the limits that were imposed by the available electrical apparatus. This cable has the merit that the covering is impervious to water for considerable periods of time. It does unfortunately give rise to slightly greater errors in the recording of the depths of fluid than enamelled wire, but even then the total error was judged to be not greater than 0·015 in. The great advantage was that its capacitance was much less than that of the enamelled wire, and the output of the electrical apparatus available was linear for only a very small range of capacity change in the circuit. The linear range was approximately ± 20 picofarads, and as the enamelled wire had a capitance of approximately 40 picofarads per inch, it was only of use for changes in depth less than one inch. Since the maximum change in depth was approximately 4 in., and the P.V.C. covered cable has a capacitance of a fraction over 10 picofarads per inch, it was the obvious choice.

It was discovered that if a very thin coat of Vaseline was smeared on the cable that the meniscus effect was considerably reduced, and it is believed that the total error in the measurements would then not be more than 0·01 in. It was essential however that the grease coating should only be the faintest smear; otherwise noticeable variations in the overall diameter of the cable would occur, and this could lead to even greater errors than before.

The cable used was manufactured by W. T. Henley's Telegraph Works Limited, and is sold under the trade name of "Equipment wire." It was tensioned between two perspex blocks, the lower block being fitted into one of a number of slots running practically the full width of the channel. These slots were cut in brass strips placed between each of the glass panels forming the bed of the channel. Thus depth gauges could be installed in the channel at approximately every 6 ft., with the first gauge 2 ft. from the channel inlet. The upper surfaces of the lower perspex blocks were set flush with the bed of the channel, and at a slot where no gauge was installed, a loose perspex block was inserted, so that a level bed surface in the channel was maintained.

The two perspex blocks were joined by strips of spring steel, which had been carefully streamlined to offer the least resistance to the flow. The cable passed through a small vertical hole in the lower perspex block, and fitted into a smaller groove in the underside of the block. This lower end of the cable had already been knotted, and a tail of approximately half an inch left beyond the knot. The knot held the cable at its lower end, and the bare end of the tail was then given several coatings of a proprietary insulating glue ("Radio Spares" cement). This was the only easily available insulant which was impervious to water. Since it was beyond the knot, no tension or vibration in the cable could then be transferred to the insulated end of the tail, which would then remain impervious to the water for at least a month, even if fully immersed for that period. The tail was then gently inserted into the small groove in the perspex block.

The other end of the cable passed through an oblique hole in a perspex cylinder and was knotted on the other side. This cylinder was attached to a threaded brass rod which was inserted in a vertical hole in the upper perspex block. A nut on the brass rod, and bearing on the upper surface of this block could then tension the cable by any required amount. This had to be done carefully as the tension had to be sufficient to avoid any vibrations that might be set up in the cable by the flow of water, and yet the tension must not be large enough to cause the cable to break. Using knots to anchor the cable would appear to be a "Heath-Robinson" expedient, but in fact was found to be infinitely preferable to the more usual methods of clips and screws. It is a positive, rapid,

and secure method of fastening the cable.

The upper end of the threaded brass rod was secured to the end of a graduated bar, which could be raised and lowered by a rack and pinion against a vernier. With this device the complete gauge frame could be raised or lowered any desired distance. The measuring scale was screwed to a frame which was clipped to the upper rail of the channel.

The free end of the cable beyond the upper knot was connected to the live terminal of a coaxial socket, the earth side of this socket being connected to one of the spring-steel stays, and thus to the water.

Thus the depth gauge and frame, when completed, could be slotted into the bed of the channel, and the only resistance to the flow arose from the streamlined stays and the wire probe. The stays were approximately 2 in. from the probe, but had no measurable effect on the water level there, even with the maximum flow available.

This design was realised after a period of trial and error and it fulfilled all the conditions that had been originally specified. The gauge must offer minimum resistance to the flow; be strong enough to survive repeated handling; allow easy and rapid replacement of the cable; ensure that the cable could be adequately tensioned; remain fixed in a position and free from vibration when recording, but be capable of being easily raised and lowered any measured distance; act as a conductor between the water and the earth side of the oscillator; and must be constructed to ensure that the gauge frame and the water were insulated from the metal filament in the cable. Fig. 4 shows the assembled gauge.

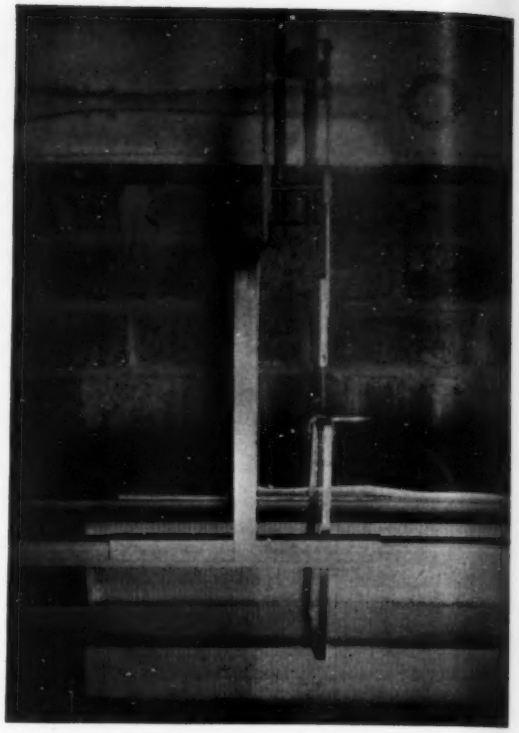


Fig. 4. Capacity type depth gauge and gauge-calibrating apparatus. (Note slot in bed of channel)

Electrical Apparatus

The coaxial socket on the gauge frame was connected by a short length of cable to an electronic oscillator. The maximum length of cable used was generally not greater than 4 ft., to minimise the effect of any stray capacitances that might have arisen. The oscillator, which was supplied by another cable at a low voltage, operated at approximately 2 Mc per sec. This high-frequency signal returned along a second conductor in the oscillator supply cable to a discriminator circuit, which was supplied with electrical energy from a power supply.

Thus as the water level changed, so the capacity of the gauge condenser varied, and this change in the oscillator circuit caused a corresponding change in the frequency of the signal relayed back from the oscillator to the discriminator. As the high-freqency signal varied, so the discriminator circuit interpreted this variation as a change in d.c. output voltage; hence any change in water level resulted in a change in the voltage output. It was very soon noticed that the relationship between the two changes in quantities was linear if the change in depth did not exceed 4 in.; or it would be more precise to say that the relationship was linear within the limits of the experimental apparatus. If the relationship of depth change against voltage output was plotted for a greater depth change than 4 in. the resulting curve had the typical N shape of a curve of this character. The lines forming the N were not completely straight, but if the depth variation was centred over the middle of the oblique stroke of the N, and did not extend too far on either side then the change could be considered linear.

The varying voltage could be recorded in several ways. It could be registered by a cathode-ray oscillograph, and the result photographed by a moving film camera. This method had the advantages that the oscillograph has its own amplifiers and is usually able to give a full-scale deflection of the signal for an output voltage of approximately 0·3 V. Also, the record can always be duplicated from the negatives. Its main drawback is that, unless another oscillograph is available, it is not known whether a record has been obtained or not until the film is developed. Since even the best-regulated electrical apparatus can and does play tricks at times, this point is of considerable importance. Secondly, handling long lengths of film when developing and printing is not a matter for the novice, with very limited equipment, however enthusiastic.

The second method of recording the changes in voltage involves the use of an electrically operated pen, the deflection of the pen being proportional to the voltage applied. Either a clockwork or electric motor can be used to slide the paper under the pen. The disadvantages of this method are the need for expensive amplifiers to boost the discriminator output voltage to a value sufficient to operate the pen, the inertia of the pen, and the uniqueness of the record. The advantages of this method, however, are a hairline and an immediately visible record.

All the experiments were recorded by the pen method using "Teledeltos" paper. The record is formed by a spark which travels between the pen and the metallic underside of the paper, singeing the paper in its path. The recorder was a single-pen Kelvin-Hughes model. The amplifier was developed in

the Sanderson Engineering Laboratories, and would give a full-scale deflection with a preamplification voltage of only 10 millivolts. This sensitivity was not required in these experiments, since the minimum supply voltage range was approximately one volt. The power supply, discriminators and oscillators were supplied by Southern Instruments Limited. Since six depth gauges were in operation at any one time, it was necessary that the record from any one could be selected at will, and this was accomplished by leading the six separate output voltages to the terminals of a rotary switch, which could connect them individually to the amplifier. To check the speed of the pen-recorder paper, the mains electricty supply could be applied to the pen via a small stepdown transformer and the rotary switch. The mains supply frequency was checked by a frequency meter, which in turn was checked by regular enquiries at the local power station. A block diagram of the apparatus is shown in Fig. 5.

An alternative method of supplying the voltage to the recorder employs an out-of-balance capacitance bridge (similar to a Wheatstone Bridge) where the output required to rebalance the bridge is proportional to the change in level. With rapidly varying changes the apparatus required to convey this information to the recorder is then as involved as that already described. Both methods have their exponents and are propably of equal merit, but the former was chosen in the experiments described in this article, solely because this type of apparatus was available. One annoying but not serious disadvantage of this apparatus was the inordinate length of time it required after being switched on (at least six hours) before successive calibration of a gauge gave consistent results. A time switch easily overcomes this difficulty.

Calibration of the depth gauge

Before the records of the wave profiles could be made, it was necessary to calibrate the depth gauges. The channel was almost filled with water, and the level was maintained by a weir gate at the outlet. The depth gauges were raised by the rack and pinion until the upper edge of the lower perspex block was at the level of the water surface. The reading on the graduated scale was noted, and the gauge was lowered by an amount equal to the initial depth of the water in the succeeding experiments, and then the electrical apparatus was "zeroed." This entailed setting the output voltage from the discriminator to a certain "negative" voltage, i.e., so that the recorder pen was offset to one side. This allowed for the fact that the water depth along the surge would always be greater than the initial depth. This setting, together with the required amplification was the result of experience. It was also found that the results from the pen recorder were non-linear beyond certain limits on the paper, and thus the record has to be maintained within these limits.

This "zeroing" completed, the pen-recorder motor was started and the depth gauge was lowered and raised by equal increments, to and from a depth approximately equal to 1·85 of the initial depth in the experiments, and then the recorder motor was stopped. The complete operation could be accomplished by one person if the recorder switch was fitted into a cable of sufficient length to reach the gauge farthest away from the recorder. The ratio 1·85 allows for the maximum depth reached at the crest of a wave before breaking, which was approximately 1·80 times the initial depth. If the deflection of the pen (in millimetres) was plotted against inches change in depth, the result in all cases was a straight line within the limits already specified.

This was repeated for each gauge in turn as the output varied from gauge to gauge for the same change in depth, and was repeated each day and at each initial depth. Afterwards, with the rotary switch set to a neutral position, the gauges were returned to their slots and the water level set to the required depth, by lowering the weir gate.

It had been ascertained by previous experiments that calibration by raising and lowering the gauge within the limits of the accuracy of measurement was equivalent to lowering and raising the water surface level.

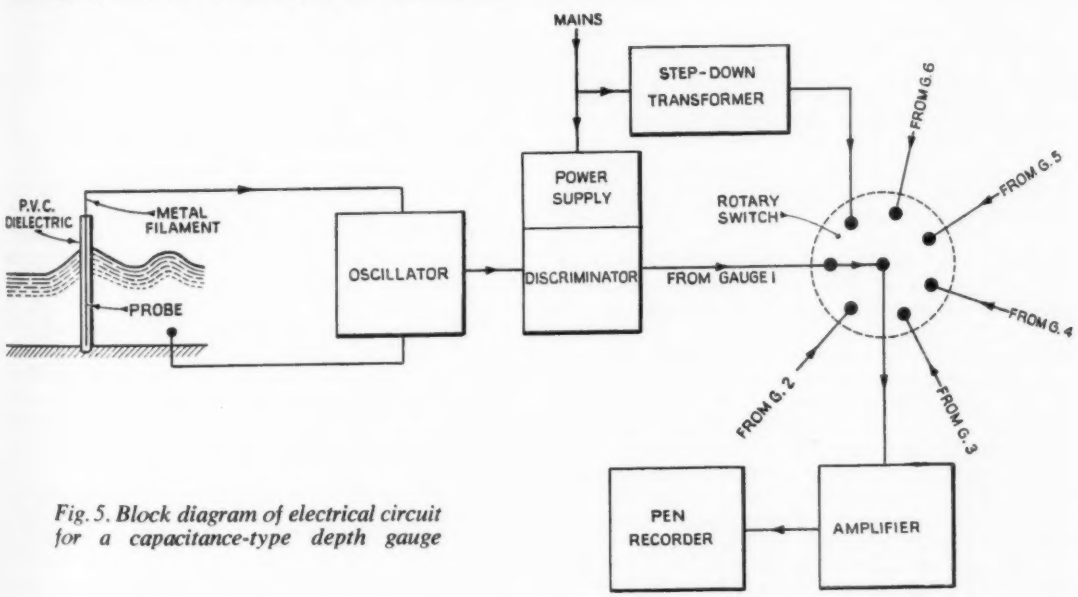


Fig. 5. Block diagram of electrical circuit for a capacitance-type depth gauge

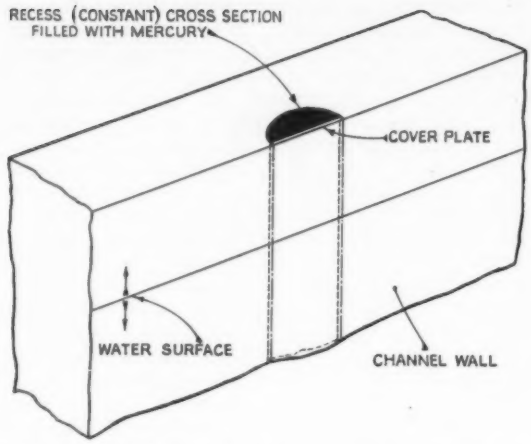


Fig. 6. Capacitance depth gauge in channel wall

Recording a surge-wave profile

After approximately fifteen minutes when the water had settled to a steady depth, the level was checked with point gauges and any necessary small adjustments made to the recording apparatus. By means of a manually operated slide valve a sudden flow of water was released into the channel, and the recorder motor was started as the surge approached the first gauge. The rotary switch had already been set to allow the recording of the change in depth at the first depth gauge. When the head of the surge was approximately 3 ft. from the next gauge, it was switched into the recorder circuit. This was repeated for the six gauges in turn, and then the recorder motor was stopped. The quantity of water flowing was obtained from the readings of a calibrated differential manometer connected to pressure tappings at either side of an orifice plate in the supply pipe. The slide valve was then shut. The water temperature, mains frequency, time of day, and date were then recorded. This procedure was called a "run."

A number of runs were made at each initial depth, with the discharge varying from a very small quantity to that required to cause the first wave to break. These runs were entitled a "series." About 23 series were recorded in these experiments under different conditions.

The tabulated information was also included on the sheet from the recorder. From the gauge calibration, the vertical displacements of any point on the wave could be determined. Since the distances between gauges and the paper speed were known, it was possible to calculate the mean wave velocity between the gauges. Interpolation or extrapolation of this information then gave the wave velocity at any gauge. Thus the horizontal distances on the wave profile from some convenient datum could be calculated, and the wave profile then plotted to scale. It is realised that using the wave velocity at the gauge as a basis of measuring wave lengths does introduce some errors, but these were usually very small and corrections could be made if necessary. The results were checked by photography.

At small initial depths, i.e., less than 2 in., the effect of ripples and of turbulence arising from the bed friction reduced the accuracy of the readings, since these effects were appreciable when compared with the amplitudes of the waves. Despite this, however, the profile could usually be plotted to a reasonable degree of accuracy as shown by comparison with the photographs.

The errors were rarely negative owing to meniscus and disturbance effects. As the initial depths increased, the percentage error in the recorded profiles was found to be very small indeed. With initial depths of 4 in. the turbulence induced by the bed friction was comparatively small, and the resulting record of the profile was extremely smooth and regular.

The results were then compared with the calculated results, and in general the agreement was most satisfactory.

Conclusions.

In conclusion a few relevant points might be considered.

A modification to the depth gauge that has been considered would avoid the use of a frame, and thus reduce resistance to the flow. If the construction of the conduit is such that the cable of the gauge can pass through a watertight packing in the bed, then the cable can be tensioned by a weight or spring. These would be underneath the conduit, and would keep the cable stretched tightly. The disadvantage of such a construction is that the gauge would have to be calibrated by raising or lowering the water level, which is extremely tedious.

This capacitance method of measuring depths might well be extended to various commercial processes where such information is required. It can be used in either open or closed channels, and if the velocity of the flow is small, much longer and more robust gauges could be used, such as the capillary tubing already mentioned. An alternative method, which does not impede the flow, is available for larger depths. The only requirement is that one wall of the conduit should be straight, although this is not strictly necessary. If the walls are not too thick and are of a non-conducting material, then they can be used as the dielectric. A thin strip of metal against the wall, and placed normally to the direction of flow acts as one conductor and the water as the other. Alternatively, a vertical slot could be cut in the wall, a glass panel sealed across the front of it, and the hollow behind the glass filled with mercury. (Fig. 6.)

The capacity depth gauge can also be used with modifications to measure variations in depth with respect to distance. An example of this is the recording of the undular profile of the flow over a broad-crested wire. Individual depths may not be as accurately measured as they could be with a point gauge, but the record is continuous with the capacity depth gauge. It only requires that the gauge be moved with a known velocity.

When recording the depth change in a flow of liquid with one of the capacity depth gauges already described the cable offers a certain resistance to the flow. The water tends to rise on one side of the cable and drop on the other side, but the net result is that the record shows a greater depth than actually occurs. This error is proportional to the diameter of the wire and to the velocity. It should be noted, however, that a surge wave moving with a velocity V will produce a smaller error than an ordinary flow with mean velocity also V, because the wave velocity is much greater than the mean particle velocity in the wave.

Different forms of capacitance gauges already have many applications in industry, for example a similar method is used to measure rapidly varying pressures. Another application measures the moisture content of grain. The apparatus is calibrated by an equivalent amount of dry grain and grain moistened with known quantities of water. The moisture reduces the capacitance of the condenser as it acts as a conductor, and a rapid and reasonably accurate estimate of the moisture content is available.

The foregoing is merely a brief survey of the different types of electrical gauges, and the design and operation of one gauge of a particular type has been emphasised. There are a number of others; for instance, in France a capacitance depth gauge has been developed on similar lines which separates transient fluctuations in level such as waves, from long-term changes in level such as tides. This gauge is very useful in model studies of wave action in tidal basins.

The channel and capacitance depth gauge described were designed by the author for an experimental programme at the Sanderson Engineering Laboratories, Edinburgh University. He wishes to thank Professor R. N. Arnold for the facilities and assistance afforded to him, and Dr. O. C. Zienkiewicz for all his help, encouragement, and good advice.

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Trade Publications

Riva-Calzoni. We have received a handsome brochure setting forth the activities of the Group formed nearly 30 years ago by Costruzioni Meccaniche Riva, Milan, founded in 1861, and Allessandro Calzoni, Bologna, founded in 1830. These two firms have behind them a long and famous history in the contruction of water turbines, pumps and other equipment for power generation, land reclamation and irrigation. As long ago as 1899 the Riva Company supplied two 3,000 h.p. turbines to Niagara, which were then the most powerful in the world, and last year this same Company built the world's most powerful Pelton sets—the two double-runner single-jet machines for Cimego. An account is given of the entire remodelling and re-equipment of the hydraulic research laboratory, and mention is made of the capacity of the works to handle not only precision engineering, such as is needed for governors, but also the very large pieces called for in the design of the largest turbines. For this heavy work special machine tools were designed and built by the Group. The brochure not only traces developments in Pelton, Francis and Kaplan turbines, but also deals with governors, control cabinets, valves, gates, pumps, hydraulic couplings, pneumatic tools and industrial machines. The brochure is in Italian but we understand that an English edition is in course of preparation.

J. M. Voith G.m.b.H., Heidenheim (Brenz), are to be congratulated on the uncommonly handsome publication they have issued to their personnel, more especially for the benefit of new-comers to their staff. It is a mere coincidence that the book appeared just at the time when the firm could have celebrated their 90th year of activity as an industrial concern. Messrs. Voith regard their birth as dating from the 1st of January 1867, the date on which J. M. Voith, who gave the firm his name, retired and handed over the business to his son. In fact, as recorded in the book, their origins go much farther back, since J. M. Voith himself had taken over from his own father, in 1825, a locksmith's workshop which had been established much earlier. There is a long step from the unassuming workshop in a backstreet of Heidenheim to the present four great establishments in Heidenheim, Bremen, Crailsheim and St. Pölten, with their modern equipment and their thousands of workers. It was through the manufacturing of paper-mill machinery, a branch of engineering in which Messrs. J. M. Voith still enjoy world-wide reputation, that the firm became interested in water turoines. Their first Jonval turbine, built as early as 1870, was followed three years later by a Francis unit which can still be seen at the German Museum in Munich; then, with the harnessing of higher heads, about 1900, came the Pelton wheels and, some twenty years later, the first Kaplan turbines, a design with which the Voith works in Heidenheim and St. Pölten have always been closely associated. The concern can now proudly look back upon an aggregate production of 16,000 water turbines of all types totalling 20 million h.p., not to mention the many steel structures and items of ancillary equipment which enter into hydro-electric developments. The production of the Voith works also extends to many other branches of engineering. We should not like to close this note without mentioning the outstanding achievements of the J. M. Voith concern in the welfare of their personnel, as described in the final section of their publication.

Longer Belt Life. Under this title the "Texrope" Drive Department of Allis Chalmers have issued a 12-page bulletin (20X6234C) to explain how the life of V-belts can be prolonged by taking a few simple precautions, which also have the advantage of increasing the efficiency of operation. Correct methods of installation are described and some common faults, frequently destructive, are portrayed.

Guyrex Steel Shuttering. A comprehensive brochure received from Guyrex Equipment Limited, illustrates and describes the advantages of their steel shuttering panels which are stocked in two widths, 9 in. and 12 in., and in lengths ranging from 4 ft. to 10 ft. The shutters and ancillary equipment are for sale or hire.

Brush High-Voltage Equipment. The Brush Electrical Engineering Co. Ltd. have issued three new publications dealing respectively with their High-Voltage Switch Fuse, Type HF 11 (No. 41014); their Extensible Outdoor Ring Main Unit, Type HF-OJ (No. 41016); and their High-Voltage Outdoor Oil Switch (No. 41017). All this equipment is A.S.T.A. certified and is fully illustrated and described in the brochures.

The Durance Project

The investigations, undertaken by Electricité de France and other authorities, to ensure flood-discharge works of adequate capacity, at Serre Ponçon, are the subject of the main section of this article by Peter O. Wolf, B.Sc.(Eng.), A.M.I.C.E., A.M.I.W.E.

PART THREE

A GENERAL outline of the comprehensive development of the Durance basin was given in Part One*, and Fig. 6 will serve to remind the reader of the hydro-electric part, although it is not possible to show on it the works for irrigation, water supply and flood protection which are as important as those for power generation.

Part Two† was devoted to a description of the proposed great dam at Serre Ponçon. Two additional illustrations, Figs. 7 and 8, are here included to re-emphasise the points made earlier on the difficulties of site exploration during the past century, and on the problems arising out of the creation of the largest artificial lake in France. The main section below will be concerned with studies of a kind to be undertaken wherever large earth dams are used to pond back upland rivers.

Design of the Flood Spillways at Serre Ponçon

Part Two† of this series has already referred to the need for the most careful estimate of the worst flood likely ever to enter the Serre Ponçon reservoir. In this matter the Serre Ponçon dam is exceptionally vulnerable, for as an earth dam it is not designed to withstand any erosion by overflowing water, or even heavy spray; and the topography of the site has made it impossible to provide a free overfall weir of great length which, with a small increase in reservoir level, would give a much larger increase in discharge capacity.

Fig. 5 of Part Two showed that the high-level flood spillway and the two bottom draw-off tunnels are all to be constructed as closed conduits through the Serre de Monge, a rocky spur which forms the left abutment of the Serre Ponçon dam. Once they are at full flow they have virtually reached their maximum discharge capacity which, therefore, had to be chosen so that the reservoir level would be kept below the normal maximum design value of 782·5 m. (the crest of the dam being at 788 m. and the freeboard 5·5 m. or 18 ft.) even with the worst inflow into the reservoir occurring at a time when the power station is shut down.

The high-level spillway tunnel will be 410 m. long, of 9·3 m. internal diameter, and fed through two automatically operated sector gates resting on concrete sills (of Creager profile) at 765 m., and through a steep bellmouthed transition. At the downstream end, the flow through the spillway tunnel will be directed into the balancing reservoir by means of a deflector bucket which, with its lip sloping from 678 to 690 m., will cause the jet to fall over a wide area so that the vast hydraulic power of approximately 3 million h.p. may be dissipated without undue damage. The design of the whole arrangement has been tested on a hydraulic scale model.

The two bottom draw-off tunnels are the same 9·3 m. internal-diameter concrete-lined structures which, as diversion tunnels, have a total discharge capacity of 1,700 to 1,800 cu. m. per sec. On completion of the main dam, each will be provided with a valve of approximately 14 sq. m. opening, downstream of the take-offs of the power tunnels, capable of discharging over 600 cu. m. per sec. under maximum head. At the discharge ends, both these draw-off tunnels will be provided with scoops (also tested by means of hydraulic models), situated at or below the water level of the balancing reservoir. The length of these tunnels is approximately 900 m., and the level of the sills at inlet 668 m. Emergency gates, operated from a house above reservoir top level, will be available for closing the inlets. The normal control gates for the bottom draw-off tunnels will be operated from a chamber immediately above the tunnels and accessible from the underground power station.

In a paper‡ presented in March, 1956, Mons. Labaye provided a justification of the spillway capacities of Serre Ponçon on the basis of a complete economic analysis. His study involved a balance of the cost of spillway and other protective works against the present value—truly enormous—of the cost of possible flood damage, and was based on a flood formula put forward in 1955 by Messrs. Morlat, Billiet and Bernier as a synthesis of all the information available on the Upper Durance. In 1956, these three authors published a further paper§ in which they provide much information on the Durance and, moreover, on the applicability of the theory of extreme values which appeals so powerfully to designers of reservoirs and flood-protection works. Their general conclusions are supplemented by a valuable paper by Mons. L. Serra‖ on the climatological and meteorological background of extreme floods.

* WATER POWER, Vol. 9, No. 4, April 1957, p. 146.
† WATER POWER, Vol. 9, No. 5, May 1957, p. 177.

‡ *Mémoires et Travaux*, Société Hydrotechnique de France, fascicule spécial 1956.

§ MM. Morlat, Billiet and Bernier, "The Floods of the Upper Durance and the Statistical Theory of Extreme Values" (in French), Int. Union of Geodesy and Geophysics, Int. Assoc. of Scient. Hydrology, Publication No. 42, *Symposia Darcy*, Vol. III, September 1956, p. 99.

‖ L. Serra, "The Meteorological Study of the Possibilities of Occurrence of Floods: Application to the Durance" (in French), loc. cit., *Symposia Darcy*, Vol. III, p. 291.

Available Information on Discharge

Daily observations of rainfall and river flow in the Durance catchment go back over half a century. Two daily river-gauging stations have been in operation near Serre Ponçon, at Ventavon (since 1920) and at Archidiacre (since 1916). For various reasons, the results of the station at Archidiacre, situated downstream of Serre Ponçon, are considered to be more accurate than the others and have been used as a basis for the flood studies for the new reservoir. Because of the influence of snow-melt on run-off, it has been the practice to separate the spring (May–June) from the autumn floods (September–November), and the maximum values of the daily mean discharges

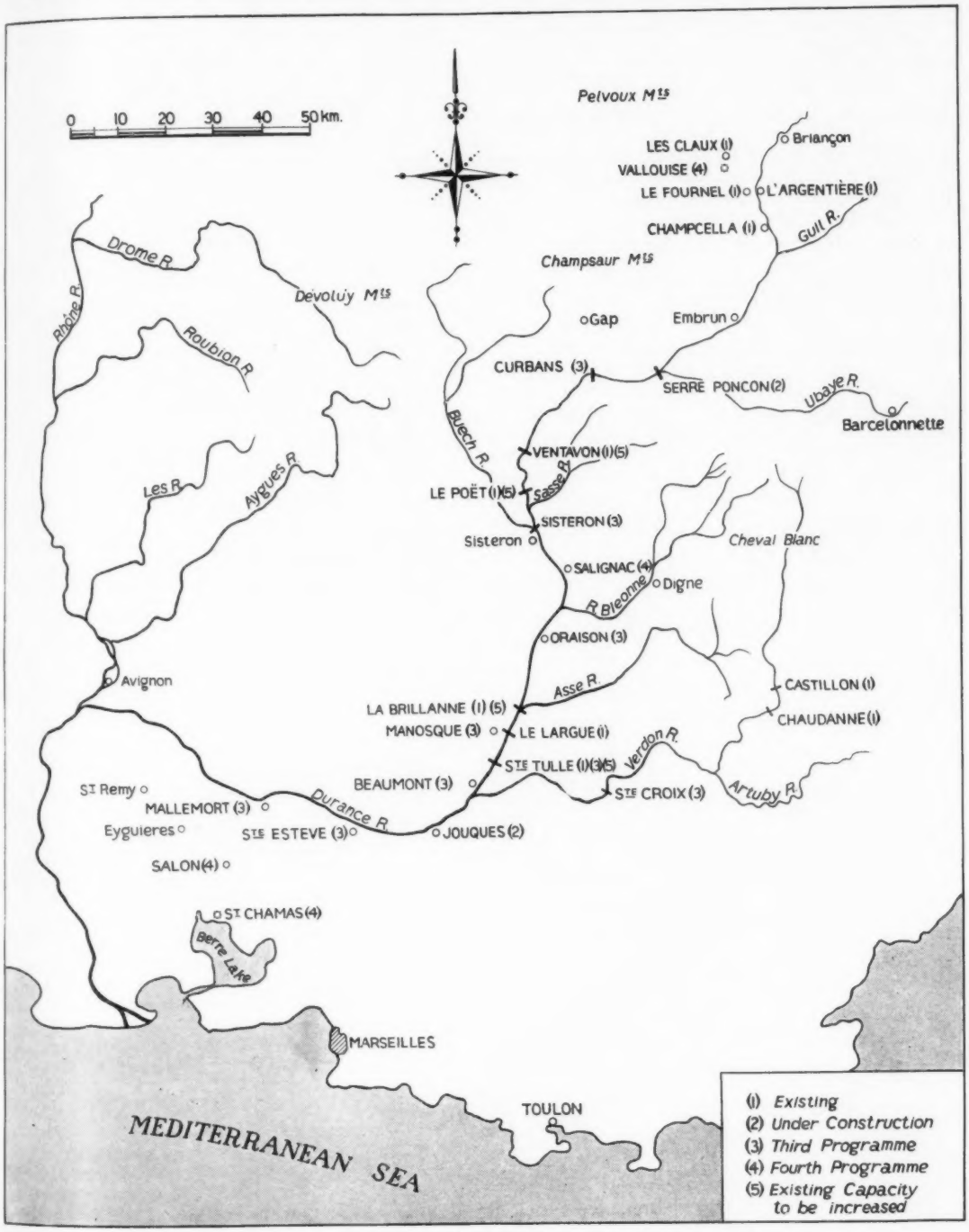


Fig. 6. Scheme of hydro-electric development of the Durance basin

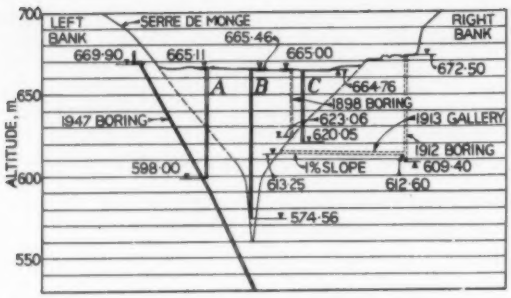


Fig. 7. Cross section of site of Serre Ponçon dam

observed in each spring and autumn have accordingly been used as the data to which the various statistical methods of analysis have been applied.

During the forty years 1916–1955, these maxima exceeded 500 cu. m. per sec. only four times in the spring (twice in May and twice in June), and the autumn maximum only once reached 500, in September.

Statistical Analysis of Discharge Data

Using normal probability theory, a curve drawn through the points representing the observed maxima would allot to a flood of 2,000 cu. m. per sec., a probability of 1 in more than 100,000 years. (It is not suggested here, or in any of the publications mentioned above, that a series of 40 annual observations will give an indication even of tendencies to be expected over the next 100,000 years, both for statistical reasons and because of the likelihood of climatic changes.) Although there have not been any serious floods during the past forty years (the maximum daily mean being recorded on 7th June, 1955, as 730 cu. m. per sec.), there is a great deal of evidence of two very heavy floods occurring during the 19th century, approximately 1,500 cu. m. per sec. on 2nd November, 1843, and perhaps 2,000 on 30th May, 1856. The meteorological causes of the 1856 flood represent a coincidence of several rare and remarkable phenomena, but their very occurrence makes the normal analysis suspect.

Keeping within shorter recurrence intervals, however, separate studies of the spring and autumn data showed the variability of the autumn floods to be greater. Although spring floods of 500 cu. m. per sec. are likely to occur more often than autumn floods of the same magnitude, floods of 1,100 would seem to be about equally likely to occur in spring and in autumn (perhaps once in a century, according to normal analysis), and further extrapolation would suggest autumn floods to yield the more dangerous extremes. In view of the experience of the past 120 years, however, it appears safer not to give much weight to such conclusions.

In recent years hydrologists have recognised with increasing clarity that straight-line extrapolation of data plotted on normal probability paper will not allot an acceptable recurrence interval to extreme floods. Professor E. J. Gumbel* has studied the application of the statistical theory of extreme values to flood series and provided evidence of some remarkably satisfactory concurrence. Gumbel's method of

* One of the best-known works is a record of four lectures published as: E. J. Gumbel, "The Statistical Theory of Extreme Values and Some Practical Applications," U.S. National Bureau of Standards, *Appl. Maths. Series*, No. 33, 1954.— On 25th April, 1957, Professor Gumbel lectured on the same subject to the Statistical Section, Department of Mathematics, University College, London. He there referred to E. J. Gumbel, "Graphical Methods of Flood Discharge Analysis" (in French), *La Houille Blanche*, November 1956, p. 709.

Fig. 8. Map of the Serre Ponçon reservoir

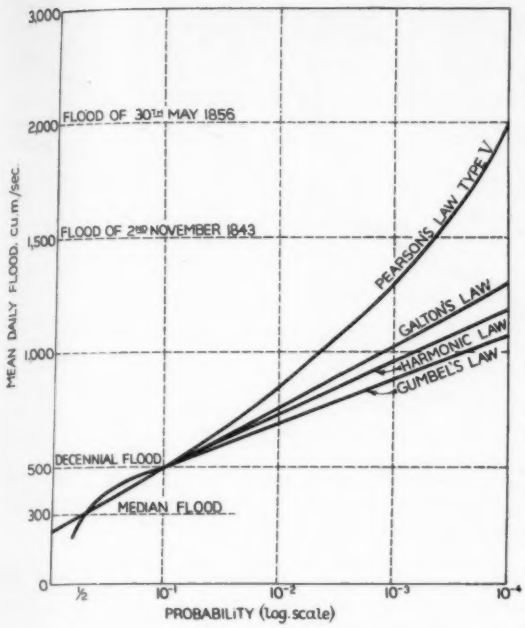


Fig. 9. Extrapolation from the annual floods observed near Serre Ponçon in 1916—1955: Comparison of four formulae

analysis, together with various others, has therefore been applied to the Archidiacre data, and the results are plotted in Fig. 9, in which it has been assumed that the median flood of probability ½ is 300 cu. m. per sec., and the ten-year flood, of probability $\frac{1}{10}$, 500. These are perhaps the limits of agreement among the exponents of various theories, but with a forty-year record it is considered reasonable to define a ten-year flood within confidence limits of 450 to 600 cu. m. per sec.

It must not be overlooked that Gumbel's theory does not suggest his frequency function to be unique; on the contrary, he stipulates that other exponential functions with a convergence more rapid than that of the lowest one plotted in Fig. 9 will satisfy the basic theory; but he has found by analysis of a large number of hydrological and other statistics (including, for example, rainfall, humidity, and temperature series of direct interest to the student of floods) that hydrological extremes are most frequently described by the function plotted as the lowest curve on Fig. 9. Gumbel's general theory, therefore, includes all three lower curves on Fig. 9. It does not, however, include what have been called algebraic functions, of the kind defined by Karl Pearson (type V) or by the French mathematician Maurice Frechet, which possess an even more rapid convergence.

There is not space here to define the mathematical functions mentioned or to discuss them in detail, but Figs. 10 and 11 are included in order to show the difficulties of flood analysis. Fig. 10 indicates that Gumbell's simple exponential formula describes the forty years' record of autumn floods on the Durance with astonishing accuracy. Had it not been for the flood of November, 1843, there would be little doubt in the designer's mind about the validity of the exponential formula. Fig. 11 shows a plot of all annual floods and indicates that even the maxima of the forty-year record (which occurred in the spring) are not accurately described by the simple exponential formula, whereas Frechet's algebraic one follows them and ascends to the value of a century ago (approximately 2,000 cu. m. per sec. in May, 1856) at a frequency which is not entirely unreasonable, perhaps once in 2,000 years.

Other Flood Estimates

Both Figs. 10 and 11 include references to empirical probability laws. These have been described, for the case of the Durance and others, by the late Etienne Halphen* who concluded that design values of floods at Serre Ponçon, based on the floods of the nineteenth century and on the forty years of recent record, might fall within the range of 2,500 to 3,600 cu. m. per sec. His method is essentially one of refined curve-fitting, and although it does not possess the sound theoretical basis of extreme-value theory, like Gumbel's, it is clearly of practical value.

Another estimate, based on a profound knowledge of the French Alps and on an unrivalled experience of flood data from all parts of the globe, has been provided by Professor Maurice Pardé, of Grenoble. He had estimated the flood of 1856 to have reached 1,800 cu. m. per sec., and he considered that double

* E. Halphen, "A Statistical Method of Analysis of Flows" (in French), International Union of Geodesy and Geophysics, Int. Assoc. of Scient. Hydrology, Rome Assembly, 1954.

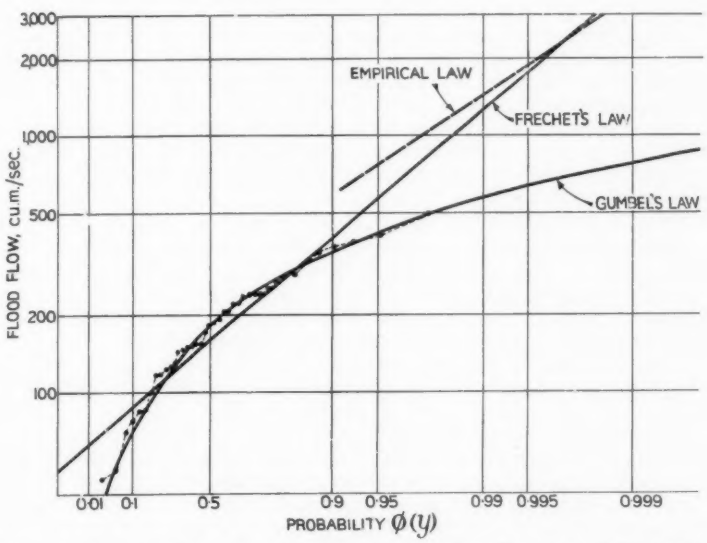


Fig. 10. Autumn floods in the Durance near Serre Ponçon, 1916—1955

that value might be discharged under extreme flood conditions.

Monsieur Serra, whose study of the meteorology of the area was mentioned earlier, is continuing his analysis without, however, expecting it to lead to a considerable modification of the estimated extreme floods.

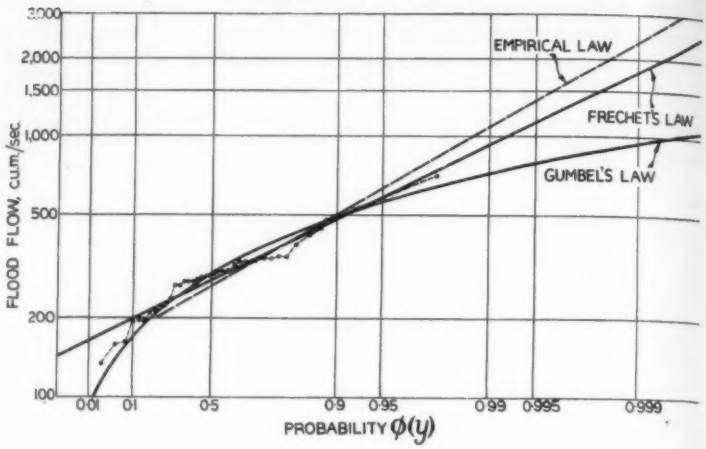


Fig. 11. Annual floods on the Durance near Serre Ponçon, 1916—1955. The straight line representing Frechet's Law has been drawn also to allow for the floods of 1843 and 1856.

Operation of the Flood Spillways at Serre Ponçon

As mentioned in Part Two, the discharge capacity of the high-level spillway will be 1,800 cu. m. per sec. when the reservoir level is at 780 m., and this capacity will increase to its virtual maximum of 2,000 with a further water-level rise to 782·5 m. With the joint capacity of the bottom draw-off tunnels at 1,200, the total flood discharge will, therefore, be 3,000 cu. m. per sec. at normal top water level, and 3,200 with 2·5 m. or more of surcharge.

Since it is in practice impossible to make even a 24-hour forecast of the peak rate of inflow or of the amount, duration or exact distribution of the total volume of inflow, the intention is to operate the flood-spillway gates so as never to discharge downstream more than the inflow rate, i.e. always to allow the reservoir surface to rise and to give protection from floods to the Middle and Lower Durance, as far as will be possible.

Margin of Safety

Floods will rarely coincide with that stage on the annual cycle when the reservoir is quite full, and the majority of floods will be completely absorbed in raising the reservoir water level: near the top, each m. of rise represents a volume of approximately 25 million cu. m. of storage, or 1% of the mean annual run-off. Power production absorbs as much as 300 cu. m. per sec., and by running the turbines continuously for suitable periods, a further margin of flood control without waste of water will be available.

Assuming, however, that the reservoir is initially full and that no water is passed through the power station, the spillway capacity of 3,000 to 3,200 cu. m. per sec. is clearly sufficient to deal with the flood inflows which have been estimated as likely maxima.

The large freeboard to the dam crest, at 788 m., provides a further substantial margin of safety. With a rise to 787 m., a storage volume above 780 m. of over 200 million cu. m. is made available which would absorb an inflow with a peak of 5,000 cu. m. per sec., even if the duration of flow exceeding 3,200 lasted as long as 48 hours. R. Maigre* has shown that a flood extending over 120 hours, with a total volume of 850 million cu. m. (30% of the average annual run-off) and two separate peaks of 5,000 cu. m. per sec., would only raise the reservoir surface level from 780 to 783 m. To expect much more severe conditions would clearly seem to be unrealistic.

NOTE.—Figs. 6, 7 and 8 are based on information supplied by Electricité de France. Figs. 9, 10 and 11 have been reproduced from the paper by Messrs. Morlat, Billiet and Bernier, with the permission of the Secretary-General of the International Association of Scientific Hydrology.

(To be continued)

* R. Maigre, "The Serre Ponçon Dam: Engineering Features of the Works," Supplement to *Travaux*, No. 247, Paris, May 1955, on the occasion of the Fifth Int. Congress on Large Dams.

Uddeholm Limited. With a view to meeting increased demands for high-quality rock-drill steels, Uddeholm Limited of Somerset House, Temple Street, Birmingham 2, England, have formed a new Mining and Civil Engineering Division to serve the requirements of the United Kingdom. This firm is the British associate of Uddeholms Aktiebolag, an old-established Swedish company which was founded as long ago as 1668 and which today controls six steelworks and many other enterprises, including some thirty or so hydro-electric stations. The chief function of the new division will be to expand the market for the Uddeholm Company's "Uddia" rock-drill steels, which have been widely employed in water-power schemes, including such notable operations as the Harrseleforsen project in Sweden and the Tokke hydro-electric plant in Norway. The new division has been put in the charge of Mr. C. G. Middup, A.M.I.Min.E., who will function as General Manager. Mr. Middup was formerly the editor of two technical journals, *Colliery Engineering* and *Mine & Quarry Engineering*, both of which are published by Tothill Press Limited, the publishers of WATER POWER.

Hackbridge and Hewittic Electric Co. Ltd. Three new publications have been received from this firm:—DB 5/4 forced-cooled transformers up to the largest sizes and voltages; DB 10/4 naturally cooled distribution transformers up to 1,000 kVA; and DB 16/1 small low-voltage transformers 100 W to 5 kVA.

A Simplified Integration Technique for Pipe-Flow Measurement

By F. A. L. WINTERNITZ*, Dipl.Ing.E.T.M., V.D.I.,
and C. F. FISCHL*, B.Sc.

Simple new methods of numerical integration have been developed for the determination of flow rates in pipes from point-velocity measurements with either pitot tubes or current meters. These methods, employing a reduced number of measuring points, are suggested as suitable for inclusion in the existing power test codes and standards of flow measurement

THE determination of pipe flow by integration of individual velocity measurements is a standard method of considerable practical importance. For large gas flows this may constitute the only available means for gauging the flux, and it is often used in preference to available methods of bulk-flow measurement when large quantities of water have to be metered. In the latter case, any simplification consistent with accuracy requirements would be welcome in view of the very substantial outlay in time and cost. Expenses incurred in efficiency tests on large water turbines, for example, are often of the order of several thousand pounds, and the evaluation frequently takes weeks to complete. Applications on a lesser scale would equally stand to benefit from an acceptable solution.

Before attempting any simplification, the existing methods for determining flow rates in pipes from individual velocity measurements must be reviewed in the light of standard specifications. It should be noted that the paper is concerned with methods of evaluation only, and that the actual overall accuracy of the measurement is not under consideration.

Standard Specifications for Pipe-flow Measurements

The settling length up and down stream of the section of measurement, the number of gauging points and their radial location, as well as the methods of evaluation are detailed in the standards of flow measurement, the power test codes and in the handbooks of hydraulic engineering. From a survey of these, the specifications for the approach length of uniform cross-section vary from 3 times[1] the pipe diameter D to a distance of $20 \times D$[2]. Variation is similarly pronounced for the downstream settling length.

From experimental evidence [3] it is known that the settling length after which a pipe-flow velocity pattern with negligible pressure gradient is obtained, varies as a function of Reynolds number and wall roughness between about 25 to 40 diameters. The number of diameters does depend essentially on the previous flow history. However, in the absence of an upstream disturbance causing flow asymmetry, a settling length of $10D$ ensures in general a velocity distribution which resembles the final one fairly closely. Nevertheless, practical considerations frequently result in shorter approach lengths being used. Measurements may therefore have to be carried out in flow which is not fully established or, more often, in flow with an asymmetric velocity distribution.

The minimum total number of gauging points for a circular cross-section varies, depending on the standard, from eighteen[1] to twenty[4]; the relation given in the former case is

$$N \geqq 3\sqrt{A} \qquad \ldots (1)$$

where: N = number of points
A = area of metering section in ft.^{2}.

If current meters are used, the number is specified[2] by

$$14\sqrt{A} < N < 25\sqrt{A} \qquad \ldots (2)$$

where: A = area of metering section in m^2.

A modification of equation (2) is at present being considered by national and international standards committees. If accepted, the proposed draft would considerably reduce the number of measuring points required in large diameter pipes, the suggested minimum number of current meter positions being thirteen.

A further requirement, applying to measurement with current meters, is that the meter nearest the pipe wall must be located at a distance y^* such that

$$0{\cdot}75 \times d \leqq y^* \leqq 0{\cdot}2(\text{m}) \qquad \ldots (3)$$

where: d = current-meter diameter
(m) = metres.

Evidently, the number of gauging points and the cross-sectional area of the metering section are correlated on an empirical basis, equations (1) and (2), and apply to any metering section selected in accordance with standard specifications. No evidence is proffered in justification for the proposed smallest numbers of measuring points per diameter nor for the variation with area. If accuracy obtainable were the criterion chosen, it may be argued that the velocity distribution will be a more decisive factor in the selec-

* Fluid Mechanics Division, Mechanical Engineering Research Laboratory, East Kilbride.

tion of the number and location of gauging points than the pipe diameter, the only possible influence of which can be via the Reynolds number and the relative roughness factor. It is on this contention that the proposed simplified integration rules have been based.

Review of Velocity-integration Methods

The objective of a velocity integration is the determination of the flow rate through the pipe. This is given by

$$Q = 2\pi \int_{o}^{R} r\,dr\,v \qquad \ldots (4)$$

where: Q = flow rate
R = pipe radius
r = local radius at which point velocity is measured
v = point velocity

Evaluation can be graphical or numerical.

Graphical Integration. As mentioned by the power test codes, [1, 2] graphical integration by planimetering can be very accurate, one part in a thousand being claimed for precision planimeters.[5] The method, however, is slow, and suffers from the inherent disadvantage that a conversion of numerical results into a graphical presentation is required before the final, and again numerical, answer can be obtained by planimetering, each stage of the process of evaluation being a possible source of error. By comparison, numerical methods of integration are speedier in operation, and permit full use of novel recording techniques.[6]

Tangential Rule. This is the most widely used of all the recommended numerical methods for the determination of the pipe-flow discharge from velocity traverse measurements.[7], [8], [9], [10] In applying the tangential rule, the cross-section of the pipe is divided into concentric rings of equal area, the innermost being a circle. Velocity readings are taken at positions on concentric circles which bisect each ring into two equal areas. The arithmetic mean of these velocities is assumed to be the mean velocity of the flux through the cross-section. Positions for six, eight and ten-point traverses are listed in Table I.

The accuracy of the tangential rule was tested for ten gauging points per diameter* by Sherwood and Skaperdas,[11] the comparison being with results obtained by graphical integration. Eighteen experimental velocity profiles, the majority with very irregular distributions, were used. In the absence of near-wall measurements, the last experimental point was generally linked to the wall by a straight-line continuation of its tangent. Since in reality the velocity in the boundary layer near the wall decreases to zero according to a power law, the graphical reference integrations gave results which must have been too high. Correctly drawn graphs should have resulted in findings less favourable than the stated average difference of plus one quarter per cent. between the ten-point tangential rule and the planimeter values. A deviation of the

* Subsequently, the number of points always refers to measurements per diameter; it is also termed a traverse.

TABLE I. GAUGING POSITIONS FOR MEAN-VELOCITY DETERMINATION WITH VARIOUS RULES

Rule, and number of positions per diameter		Distance from wall in pipe diameters					
Tangential	6	0·044	0·146	0·296	0·704	0·854	0·956
Tangential	8	0·032 0·677	0·105 0·806	0·194 0·895	0·323 0·968		
Tangential	10	0·026 0·658	0·082 0·774	0·146 0·854	0·226 0·918	0·342 0·974	
Gauss-Sherwood	3 Weights*	0·060 5	0·500 8	0·940 5			
Gauss-Sherwood	4 Weights	0·036 0·1739	0·208 0·3261	0·792 0·3261	0·964 0·1739		
Aichelen's Empirical	2	0·119	0·881				
New Empirical	3 Weights	0·081 2	0·500 1	0·919 2			
Log-Linear	4	0·043	0·290	0·710	0·957		
Log-Linear	6	0·032	0·135	0·321	0·679	0·865	0·968
Log-Linear	8	0·021 0·655	0·117 0·816	0·184 0·883	0·345 0·979		
Log-Linear	10	0·019 0·639	0·076 0·783	0·153 0·847	0·217 0·924	0·361 0·981	

* Weights are given beneath corresponding positions; to determine the mean velocity, measured values are multiplied by weights, and sum of products divided by sum of weights.

TABLE II. CURRENT-METER MEASUREMENTS: PIPE-SIZE LIMITATIONS FOR VARIOUS RULES DUE TO TEST CODE SPECIFICATION $0{\cdot}75\ d \leqq y^* \leqq 0{\cdot}2$ m.

Rule	Tangential			Gauss-Sherwood		Empirical		Log Linear			
Positions per diameter	6	8	10	3	4	2	3	4	6	8	10
Minimum wall distance, y^*, in pipe diameters $\times 10^2$	44	32	26	60	36	119	81	43	32	21	19
Minimum pipe diameter (m.)											
Type A	0·86*	1·17	1·44	0·62*	1·04*	0·31*	0·46*	0·87*	1·17	1·78	1·97
Type B	2·1	2·9	3·6	1·56	2·6	0·78*	1·16	2·2	2·9	4·5	5·0
Maximum pipe diameter (m.) ...	4·5	6·2	7·7	3·3	5·6	1·68	2·5	4·7	6·2	9·6	10·5
Minimum pipe diameter (ft.)											
Type A	2·8*	3·9	4·7	2·1*	3·4*	1·03*	1·52*	2·9*	3·9	5·9	6·5
Type B	7·0	9·6	11·8	5·1	8·5	2·6*	3·8	7·1	9·6	14·6	16·2
Maximum pipe diameter (ft.) ...	14·9	20·0	25·0	11·0	18·2	5·5	8·1	15·3	20	31	34

* Pipe diameter smaller than specified by test code, $D_{min.} \geq 1{\cdot}2$ m. (3·9 ft.).

Type A: Small current meter $d = 0{\cdot}05$ m. Type B: Standard size meter $d = 0{\cdot}125$ m.

same order, probably caused by a similar error in the graphical reference integration, is quoted in reference (10).

The tangential rule was tested by the authors using Nikuradse's published data of measurements in smooth[12] and rough[13] pipes. The velocity profiles were throughout those of fully developed turbulent pipe flow. Results in Table III are based on a comparison with volumetric measurements; those in Table IV followed from a comparison with results obtained by graphical integration, the velocity distributions of the graphs being flared into the wall according to a power law. The standard deviation for the ten-point tangential rule, on the evidence from some thirty tests, was of the order of one per cent. by comparison with volumetric measurements, and about half that for the results obtained from some fifteen tests by reference to planimeter values. The reason for such deviations is the inadequate approximation of the velocity distribution near the wall by polynomial fitting functions.

Gauss-Sherwood Method. The most accurate of the available quadrature formulae are the ones suggested by Gauss,[14] which were adapted by Sherwood and Skaperdas[11] for the integration of pipe-flow velocity distributions. An approximation to the velocity profile is found using fifth or seventh degree polynomials, depending on whether three or four gauging points are chosen per diameter. The suitability of the method was verified by the two authors,[11] with the same experimental data previously mentioned in conjunction with the accuracy of the tangential rule. It will be remembered that the results of the graphical reference integrations were considered too high.

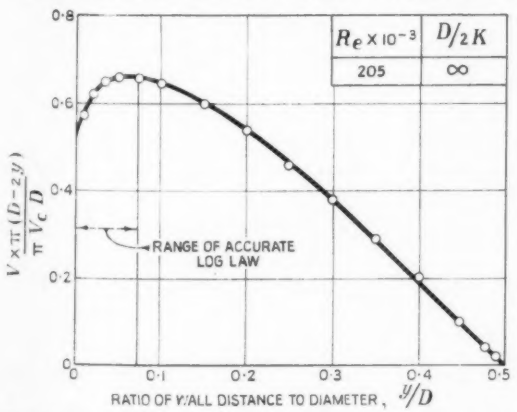


Fig. 1. Variations of flow with distance from pipe wall

For both the three and four-point traverse, the average deviations from the planimeter results were of the order of one per cent., with a maximum deviation of approximately three per cent.

Both rules were tried by the writers on some of Nikuradse's fully developed velocity profiles, comparing the results with those obtained from graphical integration, Table IV. Individual relative errors of the three-point traverse were more than plus three per cent., whilst the four-point rule yielded results accurate to within one half per cent. Caution is therefore advised in the use of the Gauss three-point method, although it has also been recommended elsewhere.[10] The four-point rule gives results to within one half per cent., and therefore is adequately accurate for most requirements, but laborious in use since complicated weights have to be carried, Table I.

Other Methods of Numerical Integration. Those devised for the integration of velocity profiles are all fairly laborious, and accuracy requirements dictate the choice of a considerable number of measuring points. Andersson[15] recently suggested a method based on the approximation of the velocity profile by quadratic polynomials, without actually applying it to the case of pipe flow. Another approach, recommended by Schulz,[16] is suitable for flow in smooth pipes, and Reynolds numbers smaller than 4×10^6. In this method, use is made of empirical corrections derived from Nikuradse's published data.

Log-Linear Rules

It is considered that the regular methods of numerical integration have proved unsuitable or impracticable, due to the steep gradient in the velocity distribution near the pipe wall. More appropriate fitting functions must therefore be used, if the accuracy required and the desired simplicity are to be obtained.

Nikuradse has shown[12, 13] that fully developed flow in rough and smooth pipes may be approximated by an equation of the form

$$\frac{V}{V_\tau} = A + B \log\left(\frac{y\ V_\tau}{v}\right) \qquad \ldots (5)$$

where: V = point velocity

$V_\tau = \sqrt{\frac{\tau_w}{\rho}}$, friction velocity

y = distance from wall

τ_w/ρ = kinematic wall shear stress

v = kinematic viscosity

A, B = numerical coefficients.

Although accurate only to within a distance of about 0·075 D from the wall,[17] the deviations occurring in the outer region of the flow are so small that equation (5) is often taken to represent the turbulent velocity profile in a pipe. Also, the range in which the velocity distribution is correctly described, contributes substantially to the total flux, Fig. 1.

TABLE III. COMPARISON OF VARIOUS INTEGRATION METHODS WITH VOLUMETRIC MEASUREMENTS FULLY DEVELOPED VELOCITY PROFILES

			NUMBER OF GAUGING POINTS PER DIAMETER AND METHOD OF INTEGRATION				
			10	4	3	2	—
			Tangential	Log-Linear	New Empirical	Aichelen's	Planimeter
Smoothness, D/2 k	Reynolds Number $\bar{V}\ D/\nu \times 10^{-3}$	Pipe Factor, $\bar{V}/V_c$	RELATIVE ERROR OF METHOD IN %				
∞	4·0	0·800	+0·6	−0·1	−0·3	−0·4	
	6·1	0·804	+1·2	+0·3	+0·6	+0·6	
	9·2	0·810	+1·1	+0·3	+0·6	+0·8	
	16·7	0·811	+3·3	+1·7	+1·6	+1·6	
	23·3	0·821	+1·0	+0·2	−0·1	0·0	
	43·4	0·832	+1·0	+0·6	0·0	−0·1	
	105	0·830	+1·0	+0·6	+0·1	+0·2	
	205	0·842	+0·7	−0·1	−0·1	+0·3	+0·2
507	22·7	0·819	+2·3	+1·7	+1·6	+1·7	
	49·0	0·832	+0·8	+0·3	+0·2	+0·2	
	106	0·844	+0·5	0·0	−0·6	−0·3	
	186	0·851	−0·8	−1·5	−1·5	−1·4	−1·3
252	21·4	0·818	+1·8	+1·2	+1·0	+1·0	
	51·0	0·837	+0·2	−0·7	−0·7	−0·3	
	103·5	0·830	+0·5	−0·4	−0·6	−0·2	
	202	0·824	+1·4	+0·7	+0·8	+0·8	+0·7
126	16·7	(0·800)	(+4·9)	(+4·2)	(+3·7)	(+3·9)	(+4·4)
	23·7	0·824	+0·9	0·0	+0·5	+0·6	
	50·5	0·830	+0·3	−0·4	−0·1	0·0	
	112	(0·807)	(+3·4)	(+2·6)	(+2·7)	(+3·0)	(+3·0)
	231	0·820	+0·4	−0·7	−0·5	0·0	0
60	15·3	0·819	+0·4	−0·6	−0·3	0·0	
	29·5	0·809	+0·8	+0·1	0·0	−0·2	
	70	(0·787)	(+3·5)	(+2·7)	(+2·8)	(+2·9)	
	116	(0·816)	(+3·8)	(+2·9)	(+3·0)	(+3·9)	(+3·8)
	271	0·796	+1·0	+0·2	+0·2	+0·3	+0·9
30	12·1	0·790	+1·2	0·0	−0·1	+0·5	
	23	0·798	+0·4	−0·7	−0·6	0·0	
	43	0·780	+1·9	+0·9	+0·3	+0·7	
	104	0·775	+2·3	+1·3	+0·7	+0·9	
	195	0·773	+1·3	+0·4	−0·6	−0·2	+0·2
15	11·3	0·753	+2·9	+1·6	+1·5	+1·7	
	22·2	0·747	+2·7	+1·3	+1·6	+1·7	
	43	0·750	+1·4	−0·1	+0·6	+1·2	
	108	0·741	−0·1	−1·4	−2·2	−2·1	
	197	0·735	+1·8	+0·7	−0·4	−0·6	+1·7
MEAN ERROR (EXCLUDING RESULTS IN PARENTHESES)			+1·1	+0·2	+0·1	+0·3	
STANDARD DEVIATION			0·9	0·8	0·9	0·8	

TABLE IV. COMPARISON OF VARIOUS INTEGRATION METHODS WITH PLANIMETER RESULTS
FULLY DEVELOPED VELOCITY PROFILES

Smoothness, D/2 k	Reynolds Number $\bar{V}\ D/\nu \times 10^{-3}$	Pipe Factor, $\bar{V}/V_c$	NUMBER OF GAUGING POINTS PER DIAMETER AND METHOD OF INTEGRATION: 10 Tangential	4 Log-Linear	3 New Empirical	2 Aichelen's	4 Gauss-Sherwood	3 Gauss-Sherwood
			RELATIVE ERROR IN %					
∞	205	0·842	+0·5	−0·3	−0·3	+0·1	−0·3	+3·4
"	1959	0·872	+0·7	+0·2	−0·1	+0·2		
"	2350	0·875	+0·8	+0·2	+0·1	+0·6		
"	2790	0·876	+0·6	0·0	+0·1	+0·6		
"	3240	0·879	+0·8	+0·3	+0·4	+0·6		
507	186	0·851	+0·5	−0·1	−0·2	−0·1	−0·3	+3·4
252	202	0·824	+0·7	0·0	+0·1	+0·1		
126	16·7	(0·800)	+0·5	−0·2	−0·7	−0·5	−0·3	+3·5
"	112	(0·807)	+0·4	−0·4	−0·3	0·0		
"	231	0·820	+0·4	−0·7	−0·5	0·0		
60	116	(0·816)	0·0	−0·9	−0·8	+0·1	−0·6	+2·6
"	271	0·796	+0·1	−0·7	−0·7	−0·6		
"	438	0·801	+0·4	−0·6	−0·5	−0·3		
"	677	0·801	−0·1	−0·5	−0·9	−0·2		
30	195	0·773	+1·1	+0·2	−0·8	−0·4	0·0	+5·4
"	372	0·782	+0·9	−0·8	−1·3	−1·0		
"	638	0·771	+0·4	−0·8	−0·8	−0·2		
15	197	0·735	+0·1	−1·0	−2·1	−2·5		
MEAN ERROR			+0·5	−0·3	−0·5	−0·2	−0·3	+3·7
STANDARD DEVIATION			0·3	0·4	0·7	0·7	0·2	1·0

For a given velocity distribution, equation (5) can be written

$$V = A^* + B^* \log (y/D) \qquad \ldots (5a)$$

where: A^*, B^* have the dimension of velocity
D = diameter.

It has been shown[18] that the velocity distribution of not fully developed flow can often be described by

$$V = A^* + B^* \log (y/D) + C^* (y/D) \qquad \ldots (6)$$

where: C^* = constant for a given velocity distribution, and has the dimension of velocity.

The basis for the proposed integration rules was the selection of metering position in each annulus such that the exact mean velocity for a particular cross-section would occur at the selected gauging points, provided the chosen logarithmic functions are an adequate representation of the considered pipe-flow velocity profiles. The mean velocity of the flow through the cross-section was to be obtained as the arithmetic mean of the individual measurements. Equation (6), in parts in conjunction with (5a), has been used to calculate the traverse position listed in Table I, details being given in the Appendix.

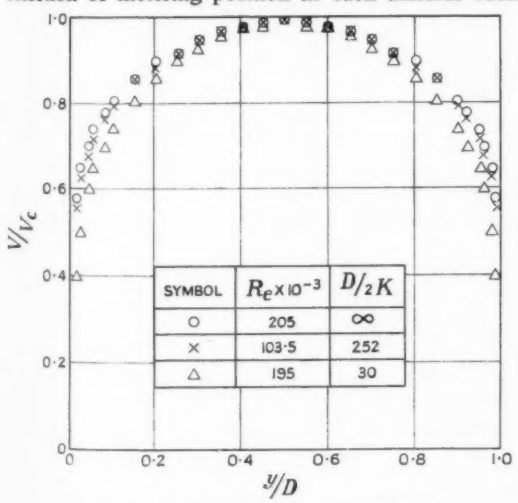


Fig. 2. Fully developed velocity profiles

Developed Pipe-flow Velocity Profiles. The log-linear rule with four gauging points per diameter was tested by application to many of the velocity profiles measured by Nikuradse in smooth[12] and rough[13] pipes, examples being shown in Fig. 2. Log-linear rules using more positions per diameter were applied to not fully developed as well as asymmetric velocity profiles, and are discussed under separate headings.

In the case of the four-point traverse, the comparison was with Nikuradse's volumetric measurements, Table III,* and with graphical integrations carried out for the purpose, Table IV. The graphs used for planimetering were extrapolated in the region of the wall in accordance with a power law.

Excluding the four doubtful tests, the mean error of the four-point log-linear traverse was less than one half per cent. and was smaller than that found for the ten-point tangential rule, Tables III and IV. Since the standard deviation is about the same for both methods,

* It is of interest that four of the volumetric measurements, in parentheses in Table III, gave results which were more than two per cent. smaller than those obtained from both the log-linear and the ten-point tangential rules. A graphical check showed three of them in better agreement with the numerical integrations than with the volumetric measurements. It has already been pointed out that some of the mean velocities given by Nikuradse were too low.[19]

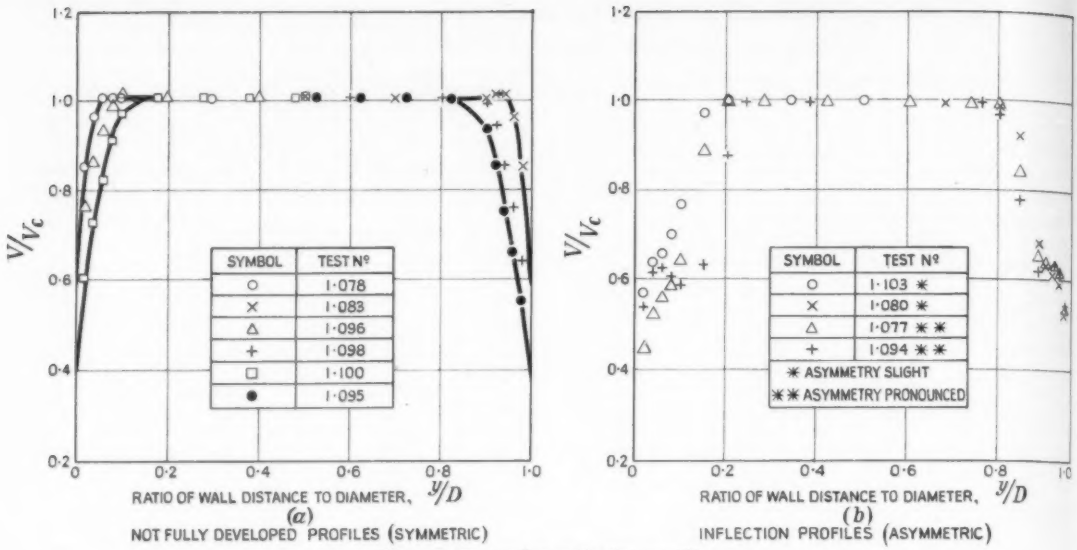


Fig. 3. Irregular velocity profiles

preference is given to the log-linear rule, using only four points per traverse.

Irregular Velocity Distributions. Log-linear numerical integration with four, six, eight and ten gauging points per diameter was applied to a variety of symmetric as well as asymmetric velocity profiles, examples being shown in Figs. 3a and 3b. The respective mean velocities were calculated previously, using the

TABLE V. COMPARISON OF VARIOUS INTEGRATION METHODS WITH 100-POINT TRAVERSE. IRREGULAR VELOCITY PROFILES

			Method of Integration, and Number of Measurement Points per Traverse					
			Tangential	Log-Linear				
			10	4	6	8	10	
Test	Pipe Factor $\bar{V}/V_c$	Reynolds Number $\bar{V}\ D/\nu \times 10^{-3}$	RELATIVE ERROR IN %					Symbol in Fig. 3
Set I: VELOCITY PROFILES SYMMETRIC, NOT FULLY DEVELOPED, FIG. 3A								
1·093	0·973	270	+1·1	+1·8	+0·9	−0·3	+0·1	
1·090	0·971	271	+1·2	+1·6	+0·8	−0·3	−0·3	
1·078	0·970	270	+1·0	+1·6	+0·4	−0·5	+0·1	0
1·083	0·970	266	+0·9	+1·2	+0·3	−0·7	0·0	×
1·104	0·959	267	+1·4	+0·9	+0·4	−0·3	+0·3	
1·099	0·946	263	+1·2	−1·2	−0·7	−0·7	0·0	
1·096	0·943	259	+1·2	−0·3	−0·3	−0·5	−0·1	△
1·086	0·921	257	+1·0	−3·4	−1·8	−0·7	−0·5	
1·098	0·907	250	+0·9	−4·0	−1·3	−0·2	−0·5	+
1·100	0·903	248	+1·0	−4·0	−1·2	−0·2	−0·6	□
1·095	0·884	242	+0·6	−5·4	−1·2	+0·5	−0·8	
√(mean square)†			1·1	2·8	1·0	0·5	0·4	
Set II: ASYMMETRIC VELOCITY PROFILES WITH INFLECTION POINTS, FIG. 3B								
1·103	—	230	+0·9	−2·2	+0·4	+0·5	+0·3	0
1·081		230	+1·1	−3·0	+0·9	+1·3	+0·1	
1·097		227	+0·6	−2·6	−0·5	+1·0	+0·1	
1·080		230	+0·8	−1·1	−1·3	+1·1	+0·4	×
1·101		223	+0·4	−0·4	−0·6	+0·9	+0·3	
*1·077		—	+1·6	−0·6	−0·2	+0·8	+0·4	△
1·076		—	+0·7	−1·4	−1·7	+0·3	+0·6	
*1·094		214	+0·5	+2·0	−1·2	+0·3	+0·5	+
√(mean square)†			0·9	1·9	1·0	0·8	0·4	

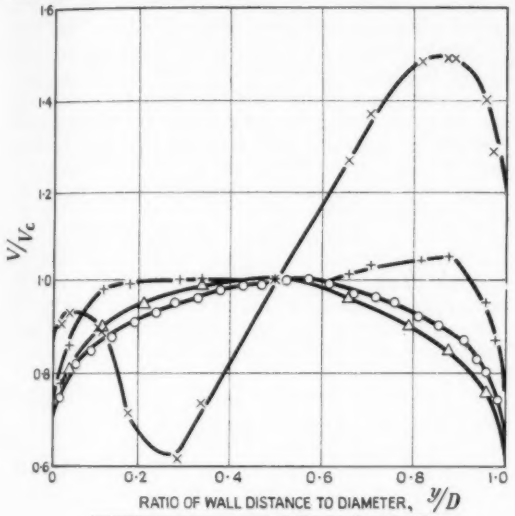
* Asymmetry pronounced.

† √(mean square) = √[Σ(Relative errors)²/Number]

tangential rule with a hundred points per diameter.

The velocity distributions of Fig. 3a are those of not fully developed flow, measured in the discharge from a contraction; those shown in Fig. 3b, featuring inflexion points as well as some asymmetry, resulted from traverses taken downstream of conical diffusers[20]. The root-mean-square values of the relative errors have been calculated as a means of comparing the various methods, definition and results being given in Table V.

For irregular velocity distributions of this kind, Table V shows that on a statistical basis the four-point log-linear rule is inferior to the ten-point tangential rule, the six-point traverse is equivalent, whilst the eight and ten-point rules are superior. The choice of a log-linear ten-point traverse results in a mean-square error of the order of one half per cent., or approximately half that of the standard ten-point tangential rule.



SYMBOL	TEST	DIAMETER FT	REYNOLDS NUMBER
×	A	5·2	4×10^6
+	B	13·0	3×10^6
O	C	4·3	$9{\cdot}8 \times 10^6$
△	E	5·4	3×10^6

Fig. 4. Asymmetric velocity distributions ascertained by field tests

Field Tests with Asymmetric Velocity Distributions. Practical considerations sometimes force a departure from standard specifications in the selection of the metering position. As a result, velocity distributions may be asymmetric and distorted. The suggested rules of integration, and empirical methods discussed in the following, have been applied to the results of five field tests available to the writers, one of them in published form.[21] Details and results are given in Table VI, whilst Fig. 4 illustrates the velocity distributions on one diameter.

Comparison was with graphical integration results, and it is seen from Table VI that the four and six-point log-linear traverse gives answers well within one per cent.; in fact, six gauging points ensure an accuracy of better than one half per cent. Of the empirical methods, the proposed three-point traverse is superior to that recommended by Aichelen, whilst both are inferior to the log-linear rules.

Although the available evidence is as yet inadequate to suggest a definite number of gauging points in the case of asymmetric velocity distributions, it is tentatively proposed on the strength of the results in Table VI, that in most cases a six-point log-linear traverse will ensure results which are adequately accurate for most purposes.

Velocity Determination by Empirical Methods from very few Gauging Points

One of the advantages of a measuring technique using few traverse positions is the possible extension of the range of application to small pipes. This may prove of importance where current meters are to be used in penstocks of small diameter.

The determination of the mean velocity from a single centre-line reading is accepted by some standards.[7] The ratio of the mean to this centre-line velocity, which depends on Reynolds number and wall roughness, is known from published data.[12, 13, 22] Its use is conditioned by the requirement that the velocity profile must be that of fully developed pipe

TABLE VI. COMPARISON OF VARIOUS INTEGRATION METHODS WITH PLANIMETER RESULTS
ASYMMETRIC FIELD TESTS

Test	Number of Gauging Points per Diameter and Method of Integration: RELATIVE ERROR IN %				Cause of Asymmetry	Distance of Metering Plane in D	Reynolds Number $\bar{V}D/\nu \times 10^{-6}$	Diameter D		Pipe Factor $\bar{V}/V_c$
	2 Aichelen's	3 New Empirical	4 Log-Linear	6 Log-Linear				m.	ft.	
A	+6·8	+3·6	−1·0	+0·3	Pump	6 downstream	4	1·60	5·2	1·123
B	+4·3	+1·3	−0·8	−0·2	Bend	11 downstream	3	3·98	13·0	0·977
C	−0·8	−0·6	+0·1	−0·2	Butterfly valve	(?) downstream	9·78	1·30	4·3	0·880
D	+0·4	+0·2	0	0	Bend and Valve	100 upstream	2·44	1·65	5·4	0·875
E	+0·3	+0·4	−0·1	0	Bend and Valve	100 upstream	3·01	1·65	5·4	0·873

flow. Hence a settling length of the order of fifty times the diameter is ideally required.

The determination of the mean flow in a pipe from a single reading was proposed by Aichelen,[23] similar rules having previously been mentioned by Ott[24], and by Colebrook and White.[25] The position specified by Aichelen as a result of empirical analysis is 0·119D; by comparison, the log-linear rule with two points per diameter yields 0·112D, the close agreement being noteworthy.

Aichelen used Nikuradse's data[12, 13] to check on the accuracy of his results, which was within one per cent. for flow in smooth pipes, but deteriorated to 2·5 per cent when used on rough-pipe data. As the results in Tables III and IV show, Aichelen's technique, when applied by the authors to data from the same source gave a mean error of less than one per cent., with a scatter comparable to both that of the ten-point tangential and the four-point log-linear rule. The latter seems therefore the most attractive method for determining the mean velocity of symmetric, developed pipe flow, since it is inherently independent of Reynolds number.

Preston and Gregory[26] suggested and investigated a very similar technique. Velocities were measured on four radii at the exact quarter-radius distance from the pipe wall; their mean was then multiplied by an empirical correction factor to obtain the true mean velocity. Results obtained in a metering plane located at least fifteen diameters downstream of a sharp-angle bend, were found to be accurate to within $\pm 0{\cdot}6$ per cent.

An empirical approach consisting of a three-point traverse, Table I, was evolved by the authors using Nikuradse's measurements, and applied to a number of his traverses, Tables III and IV. The mean error is within one half per cent., and the scatter is comparable to the other methods of integration. Applied to the few available asymmetric velocity distributions, Table VI, the proposed three-point traverse is an improvement on the results obtained with Aichelen's rule.

Current-Meter Traverses

If current meters are used, the standard specification[2] for the position of the meter nearest the pipe wall, equation (5), must be fulfilled. Hence for a given size of meter there exist for each of the integration rules minimum pipe diameters and these have been listed in Table II. Although the power test codes stipulate current meters with diameters larger than 100 mm. only, small current meters[27, 28] have also been included, to show the extension of the range to smaller pipe diameters which their use makes possible. Account has been taken of the minimum interference-free distance between meters,[28] the smallest pipe sizes resulting for Aichelen's empirical rule.

Conclusions

1. A method of numerical integration of pipe-flow velocity profiles has been proposed, which is based on the determination of the exact mean of a function of the form

$$A^* + B^* \log (y/D) + C^* (y/D)$$

Termed the log-linear law, it has been developed without taking account of the actual accuracy of measurement.

In addition, an empirical integration rule using three points per diameter has been suggested. For these and other integration rules, traverse positions have been tabulated.

2. When applied to some forty fully developed velocity profiles of flow in smooth and rough pipes, the mean relative error of the new four-point log-linear rule was less than one half per cent. By comparison, the standard ten-point tangential rule overestimated the mean velocity in all cases by approximately one per cent.

The empirical integration methods with two or three points per diameter gave results well within one half per cent., and therefore seem suitable for use with developed velocity profiles.

3. In the case of not fully developed and velocity profiles with inflection points, the root-mean-squares of the relative errors were larger than two per cent. for the four-point log-linear rule, about one per cent. for the log-linear rule with six points, and of the order of one half per cent. for eight and ten measuring points per diameter. Since in comparison the accepted ten-point tangential rule was in error by approximately one per cent, there would seem to be evidence that it may be replaced, with the same accuracy, by the six-point log-linear rule.

4. For asymmetric velocity profiles, the four and six-point log-linear rules gave results to within one, and one half per cent. respectively. The empirical three-point method was found more accurate than the two-point method on the limited number of results tested, but both methods are substantially inferior to the log-linear rules.

5. Of the previously recommended integration rules, only the four-point Gauss rule seems adequately accurate, but is considered inconvenient in use. Other methods require for an accuracy comparable to that of the log-linear rules an excessive number of gauging positions.

6. The integration rules proposed in this paper are suitable for use with current meters as well as pitot tubes. They are proposed as suitable alternatives for inclusion in the power test codes and standards of flow measurement in the form in which they are given in Table I.

Appendix—Derivation of the Log-Linear Gauging Positions

The calculation differs somewhat for an even and for an odd number of traverse positions per radius. In the case of even numbers, the cross-section is divided into concentric rings of equal area, the innermost being a circle, with two positions specified for each ring. For an odd number of positions, the outermost ring has half the area of the others. In this ring, one point only is specified, and the log-linear law, equation (6) is replaced by the log law, equation (5a).

Log-Linear Rule for an Even Number of Gauging Positions Per Radius

For $2n$ gauging positions per radius, or $4n$ points per diameter, division of the cross-section is into n equal area rings. In each ring two positions have to be selected such that the mean of the velocities obtained from these two positions is exactly equal to the mean in the ring when the velocity distribution is of the form

$$V = A^* + B^* \log (y/D) + C^*(y/D)$$

for any A^*, B^* and C^*. Hence exact mean values

will also result for log $(2y/D)$ and $(2y/D)$.

If log b_m and a_m are the mean values of log $(2y/D)$ and $(2y/D)$ respectively in the mth ring from the wall, the two gauging positions should be such that

$$\tfrac{1}{2}[\log\ (2y_m{}^1/D) + \log\ (2y_m{}^2/D)] = \log\ b_m$$

and

$$\tfrac{1}{2}[2y_m{}^1/D + 2y_m{}^2/D] = a_m,$$

where $y_m{}^1$ and $y_m{}^2$ are distances from the wall corresponding to the first and second gauging position in the mth ring. Hence

$$2y_m{}^1/D \times 2y_m{}^2/D = b_m{}^2$$

$$2y_m{}^1/D + 2y_m{}^2/D = 2a_m.$$

Therefore at the two positions

$$(2y/D)^2 - 2a_m\ (2y/D) + b_m{}^2 = 0$$

$$\therefore \quad y_m{}^1/D = \tfrac{1}{2}[a_m - \sqrt{(a_m{}^2 - b_m{}^2)}]$$

and

$$y_m{}^2/D = \tfrac{1}{2}[a_m + \sqrt{(a_m{}^2 - b_m{}^2)}]$$

Using these formulae, the position can be calculated for known values of b_m and a_m, which in turn are given by

$$\log\ b_m \frac{\pi D^2}{4}\frac{1}{n} = \int_{(1-m/n)\pi D^2/4}^{(1-m/n+1/n)\pi D^2/4} \log\ (2y/D)\ d[\pi(D-2y)^2/4]$$

and a_m

$$\frac{\pi D^2}{4}\frac{1}{n} = \int_{(1-m/n)\pi D^2/4}^{(1-m/n+1/n)\pi D^2/4} (2y/D)\ d[\pi(D-2y)^2/4]$$

hence $\log\ b_m =$

$$n\int_{1-m/n}^{1-m/n+1/n} \log\ (2y/D)\ d[(1-2y/D)^2]$$

and $a_m =$

$$n\int_{1-m/n}^{1-m/n+1/n} (2y/D)\ d[(1-2y/D)^2]$$

Setting $1 - 2y/D = t$, it follows that

$$\log\ b_m = n\int_{1-m/n}^{1-m/n+1/n} \log\ (1-t)\ d(t^2)$$

$$= n\int_{m/n}^{(m-1)/n} -\log\ (1-t)\ d(1-t^2)$$

and $a_m =$

$$n\int_{1-m/n}^{1-m/n+1/n} (1-t)\ d(t^2)$$

$$\therefore \quad \log\ b_m = n\ [-\log\ (1-t)\ (1-t^2) - \int \frac{-\log\epsilon}{1-t}\ (-dt)\ (1-t^2)]_{t^2\to 1-m/n}^{1-m/n+1/n}$$

$$= n\ [-(1-t^2)\ \log\ (1-t) - (\log\epsilon)\ (t+\tfrac{1}{2}t^2) + \frac{3}{2}\log\epsilon]_{t^2\to 1-m/n}^{1-m/n+1/n}$$

$$\log\ b_m = n\ [-(1-t^2)\ \log\ (1-t) + (\tfrac{1}{2}\log\epsilon)\ (1-t)\ (3+t)]_{t^2\to 1-m/n}^{1-m/n+1/n}$$

and $\quad a_m = 1 - n\ [(2/3)\ t^2 t]_{t^2=1-m/n}^{1-m/n+1/n}$

Log-Linear Rule for an Odd Number of Gauging Positions Per Radius

For $2n-1$ gauging positions per radius, $4n-2$ per diameter, the outermost ring has only half the area of the others. The single position in this ring should give the exact mean for the log-formula

$$\log\ (2y_1/D)\ \frac{\pi D^2}{4(2n-1)} = \int_{[(2n-2)/(2n-1)]\pi D^2/4}^{\pi D^2/4} \log\ (2y/D)\ d[\pi(D-2y)^2/4]$$

where y_1 is the distance from the wall of the first position. Therefore, log $(2y_1/D)$ is given by a formula similar to that for log b_m in section I of the Appendix. Integration yields

$$\log\ (2y_1/D) = (2n-1)\left[\frac{1}{2n-1}\ \log\ (1-\sqrt{\frac{2n-2}{2n-1}}) - (\tfrac{1}{2}\log\epsilon)\ (1-\sqrt{\frac{2n-2}{2n-1}})\ (3+\sqrt{\frac{2n-2}{2n-1}})\right]$$

For the measurement positions in the inner rings, the procedure is similar to that outlined for an even number of points. The formulae for the means are

$$\log\ b_m = \tfrac{1}{2}(2n-1)\ [-(1-t^2)\ \log\ (1-t) + (\tfrac{1}{2}\log\epsilon)\ (1-t)\ (3+t)]_{\sqrt{[2(n-m)/(2n-1)]}}^{\sqrt{[2(n-m+1)/(2n-1)]}}$$

and $a_m = 1 - \tfrac{1}{2}\ (2n-1)$

$$\left[\frac{2}{3}\ t^2 t\right]_{\sqrt{[2(n-m)/(2n-1)]}}^{\sqrt{[2(n-m+1)/(2n-1)]}}$$

The corresponding gauging positions are deduced from these means as for an even number of positions.

Acknowledgment

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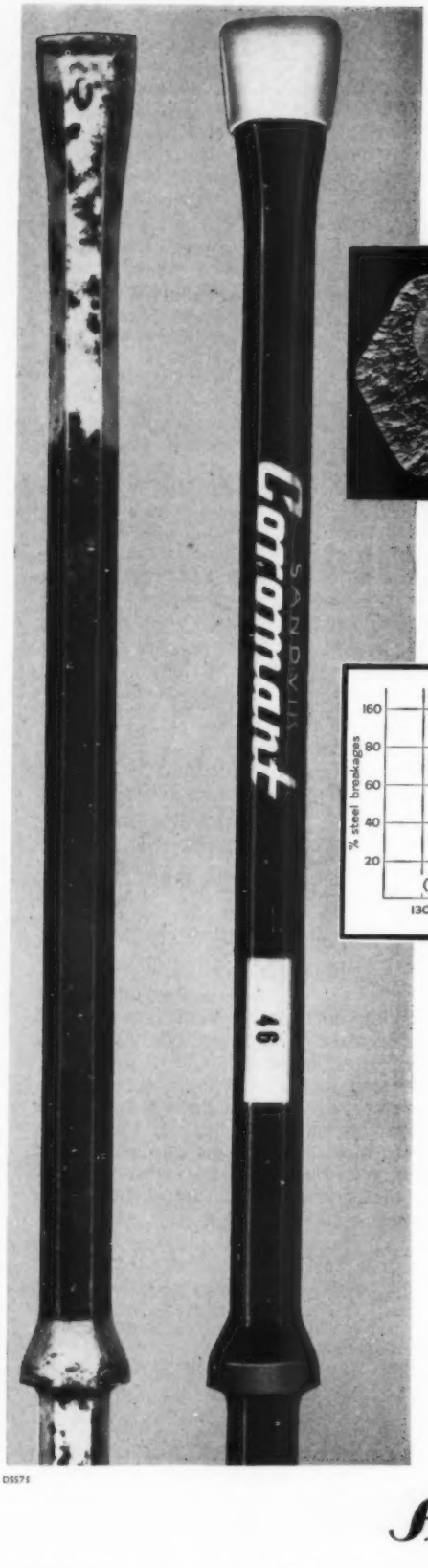
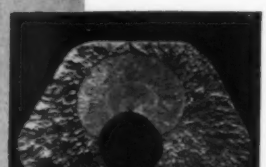


DS575

SR—the new Sandvik Coromant treatment gives steels 50% longer life

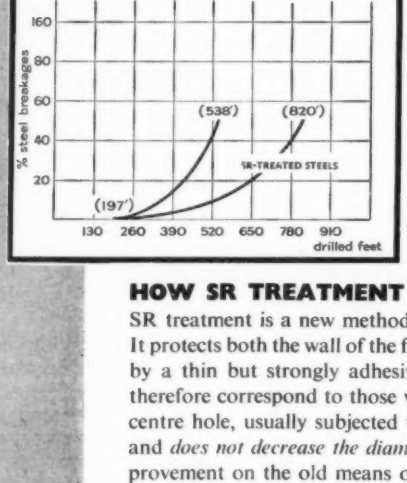


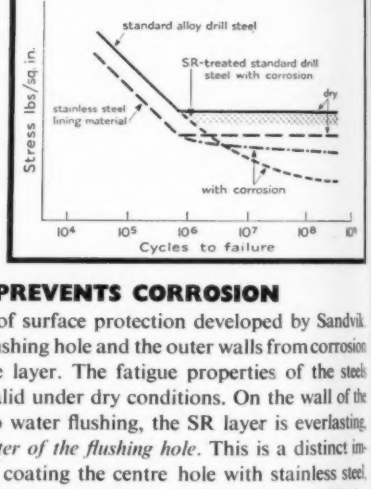
Corrosion invariably lessens the fatigue strength of drill steels and causes earlier breakages. The breakages start from corrosion craters and indentations, around which ever-widening areas of worn surfaces are formed. (See picture of cross-section.)

The main cause of corrosion is moisture—so effective protection of the steels against moisture during both storage and actual drilling eliminates corrosion and gives longer life to the steel. This protection is given to Coromant integral steels by the SR treatment, a new exclusive Sandvik feature.

PRACTICAL TESTS GIVE CONCLUSIVE PROOF

The diagram on the left shows clearly that SR-treated steel rods have a 30%–50% longer working life than those of untreated steels. Figures are based on extensive practical tests, which resulted in an average drilling depth of 820 ft. for SR-treated steels as against 530 ft. for untreated steels, before rod breakage.





HOW SR TREATMENT PREVENTS CORROSION

SR treatment is a new method of surface protection developed by Sandvik. It protects both the wall of the flushing hole and the outer walls from corrosion by a thin but strongly adhesive layer. The fatigue properties of the steels therefore correspond to those valid under dry conditions. On the wall of the centre hole, usually subjected to water flushing, the SR layer is everlasting, and *does not decrease the diameter of the flushing hole*. This is a distinct improvement on the old means of coating the centre hole with stainless steel, which lessens the diameter of the flushing hole, prevents full and effective flushing, and thus diminishes the drilling rate. Moreover, stainless steel has a lower fatigue strength than standard alloy drill steel.

Sandvik Coromant drill steels have been closely developed together with Atlas Copco rock drills. This drilling combination has for years been the most widely used in the world.

Sandvik Coromant integral steels, extension steels and detachable bits are supplied by Atlas Copco. It is the largest group of companies specialising solely in the development and manufacture of compressed air equipment. It embraces Atlas Copco companies or agents manufacturing or selling and servicing Atlas Copco equipment in ninety countries throughout the world. For further details of the equipment featured here, contact your local Atlas Copco company or agent, or write to Atlas Copco AB, Stockholm 1, Sweden.

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Properties of Lithium Base Greases

A new development in their well-known range of Lithium base greases, based on Lithium 12 Hydroxy Stearate is announced by Shell-Mex and B.P. Ltd.

Lithium base greases of this type have outstanding mechanical stability and can be used in a great variety of applications, particularly ball and roller bearings, over a wide temperature range replacing numerous conventional greases often used in the industrial and marine fields. Long "shelf-life" and resistance to drying out and hardening in service are special features of these greases. They also possess good resistance to water.

There are many instances, however, where water can gain access to bearings in service and most greases offer little protection in these circumstances. Lithium base greases of the Shell Alvania range now contain a powerful water soluble corrosion inhibitor which confers outstanding anti-corrosion properties under these arduous conditions.

This is one of the advantages achieved in the more recent research and development work carried out by Shell Research Ltd. at Thornton Research Centre on greases of this type. Other advantages include even greater oxidation stability under both dynamic and static conditions than has been achieved hitherto resulting in the service life of a charge of grease being extended in many cases.

Asea-Brush Agreement. It has been announced that the Asea Company, of Vasteras, Sweden, and the Brush Group Ltd., of London, have made an arrangement whereby the latter will acquire the equity of Asea Electric Limited, of Walthamstow, London, also trading as Fuller Electrical & Manufacturing Company Limited. It is intended that the present activities of these two companies, including exclusive representation for Asea Sweden in the United Kingdom, will continue exactly as before but under the name of Fuller. By a licence agreement between the Fuller Company and Asea, Sweden, the former will be assured for a minimum period of ten years' full participation in the technical knowledge, research and development of the Asea group of companies within the field in which the Fuller Company will be active as electrical plant manufacturers. Mr. Ian Morrow and Sir George Briggs, directors of the Brush Group, will become chairman and vice-chairman, respectively, of the company. Mr. Nils Plahn will remain managing director and there will be no change in the present management.

Hydraulic Measuring Instruments. Bearing this title Ets. Neyrpic of Grenoble have issued a brochure which illustrates and describes an exceedingly full range of measuring instruments, mostly designed by the Neyrpic organisation but some manufactured under licence. These include various types of instrument for measuring fluid velocity, liquid discharge, soundings, water level, water properties, water pressures and displacement.

Submarine Power Cable. Bulletin No. 18, published by British Insulated Callender's Cables Ltd. (B.I.C.C.) gives a full and interesting account of the submarine cable which forms the power link between Vancouver Island and the Canadian mainland.

Level Controls for Solids. Elcontrol Limited, who have for some time been manufacturing electronic level control equipment for use with conductive liquids, have now produced a range of level controls for use with free-flowing solids such as stone aggregates, sand and gravel.

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With an Atlas Copco BBD 11 you have a drill that can tackle practically any small-scale drilling job. By replacing several specialised drills the BBD 11 cuts costs considerably on mining and construction jobs. Instead of a large number of drills lying idle most of the time, one BBD 11 can undertake all their work. It can be used for boulder blasting, trimming ceiiings and walls after blasting, and plug-hole drilling in concrete; for timbering, terracing, trenching and prospecting. It will drill holes for a wide variety of purposes, including the fixing of ventilation tubes or electric wires, aligning the centre line in drifts and tunnels, and the installation of reinforcement netting for spraying tunnel walls with concrete. It is easily converted into a scaler and in this role rotation is eliminated. It's light, it's fast—and it's economical too. It can weigh as little as 18¼ lbs, sink a 12″ hole in 30 seconds, and use only 44 to 53 cubic feet of air a minute.

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For further details of the equipment featured here, contact your local Atlas Copco Company or Agent, or write to Atlas Copco AB, Stockholm 1, Sweden, or Atlas Copco (Great Britain) Limited, Beresford Avenue, Wembley, Middlesex.

1. *Boulder blasting with the BBD 11 LH.*
2. *Plug hole drilling in concrete.*
3. *Scaling with the BBD 11 LTS. Note the convenient leg support.*
4. *Survey hole drilling using the BMK 11 pneumatic pusher.*

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St. Lawrence Power Project

LAST month Dr. Otto Holden, Chief Engineer of the Hydro-Electric Power Commission of Ontario, presented a paper on the power aspects of this great waterway to The Royal Society of Arts, Adelphi, London. The St. Lawrence, he pointed out, formed a continuous stream 2,400 miles in length and drained an area of half a million square miles, and the intervention of the Great Lakes formed a vast regulating basin which kept the flow exceedingly uniform. This uniformity of flow, averaging 240,000 cusecs, combined with the drop in the river bed gave rise to very favourable power potentialities. At Niagara there was a drop of 326 ft. and between Lake Ontario and Montreal there was a total fall of 225 ft. These, combined with other minor drops of between 8 and 21 ft., gave rise to a power potentiality of some five million h.p., of which three million was in Quebec, one million in New York State and one million in Ontario.

The development in the International Rapids Section, he went on to say, would provide an installed capacity of 2,200,000 h.p. which would be shared equally between New York State and Ontario. At Iroquois a control dam was being built across the river consisting of 32 sluices, each 50 ft. wide, to regulate the flow from Lake Ontario. The major works would consist of the Long Sault dam and power station which would take the form of an arc 2,900 ft. long, 100 ft. high, and would be equipped with 30 steel gates, each 50 ft. wide and 30 ft. high, and capable of discharging a flow well in excess of any recorded maximum. Care has been taken to locate the power house so that it is bisected by the international boundary; in this way the Ontario section of the power house is entirely in Canadian territory and the New York section in United States territory.

Each power house would contain 16 generating units of 75,000 h.p. capacity, and both are being designed as semi-outdoor type structures, that is, the superstructure over the generating room has been eliminated. In its stead, each unit is being provided with a removable cover and will be serviced by a 300-ton capacity crane which will travel the full length of the power house. The upstream and downstream sides of this crane would be permanently housed in, while the other two sides would be provided with folding doors. When necessary to dismantle a unit for major repairs the crane would be moved over the unit, the unit cover, which is split on the unit centre line, would then be moved clear of the unit and the movable doors on the crane closed. In this way the unit would have protection while being dismantled, and any parts requiring major maintenance would be carried by the crane to the maintenance building at the shore end of the structure. This building would also house the control room, administration offices and reception centre.

Channel improvements associated with the power project, costing in the neighbourhood of $75 million, were being carried out for two purposes. First, to secure a maximum velocity of 4 ft. per sec. in all channels used for navigation and a maximum velocity of 2-1/4 ft. per sec. for winter flows in order to secure an ice cover.

At this point Dr. Holden said a word about the ice situation on the St. Lawrence. The ice encountered was largely of the type known as frazile—that is, ice in the form of needle-like crystals which occurred when the water has chilled below the freezing point but was in so disturbed a condition that these crystals could not coagulate on the surface to form an ice sheet. They then become mixed with the water and were carried downstream, and when they come in contact with the surface sheet ice, if the velocity is sufficiently high, they were carried underneath and form hanging dams obstructing the flow. To avoid the formation of this type of ice in large quantities sufficient excavation work was being carried out at all points, except in the Galop Rapids, to secure a velocity at which this ice would lodge against any barrier and an ice cover formed by "building" or "packing."

To assist in the design of the necessary channel enlargements and also in the location of the various dams and unwatering structures, models were constructed accurately to scale. These models covered three sections of the river representing in all a length of 36 miles and they reflected within one-tenth of a foot the water levels in the prototype for all flows. In addition to assisting in the determination of the distribution of flow after channel enlargements, where the river was divided by islands into several channels, they had proved most useful in the determination of the effects of various construction operations and resulted in very material savings in the cost of the project.

Indian Factory for Consolidated Pneumatic

The Consolidated Pneumatic Tool Co. Ltd. of London and Fraserburgh, has formed a new company under the title of the Consolidated Pneumatic Tool Co. (India) Private Ltd. The new company, which came into being on February 28, 1957, has a Directorate comprising Mr. G. J. Coffey, Mr. Norman Readman and Mr. V. Nimbkar (Managing). It will operate in Bombay where new factory premises will be constructed. The primary purpose behind this development, is the manufacture of CP products within the country, and it is expected that operations will be in full swing by the end of this year. Consolidated Pneumatic products scheduled to be made in India are pneumatic tools, rock drills and demolition tools, pumps, and air compressors. The new Company will be complementary to the Consolidated Pneumatic branches in Bombay and other Indian cities, as the entire sales and service organisation will continue without change, and will handle CP products of the British, Indian and U.S.A. factories.

Cable Order. An order for aluminium conductors to the value of over two million dollars has been placed by the Quebec Hydro-Electric Commission in Canada, with the Aluminum Wire and Cable Co. Ltd., Swansea. The conductors will be used for the 300 kV transmission lines from Bersimis to Baie Comeau, and for the eastern part of the 300 kV line from Bersimis to Montreal.

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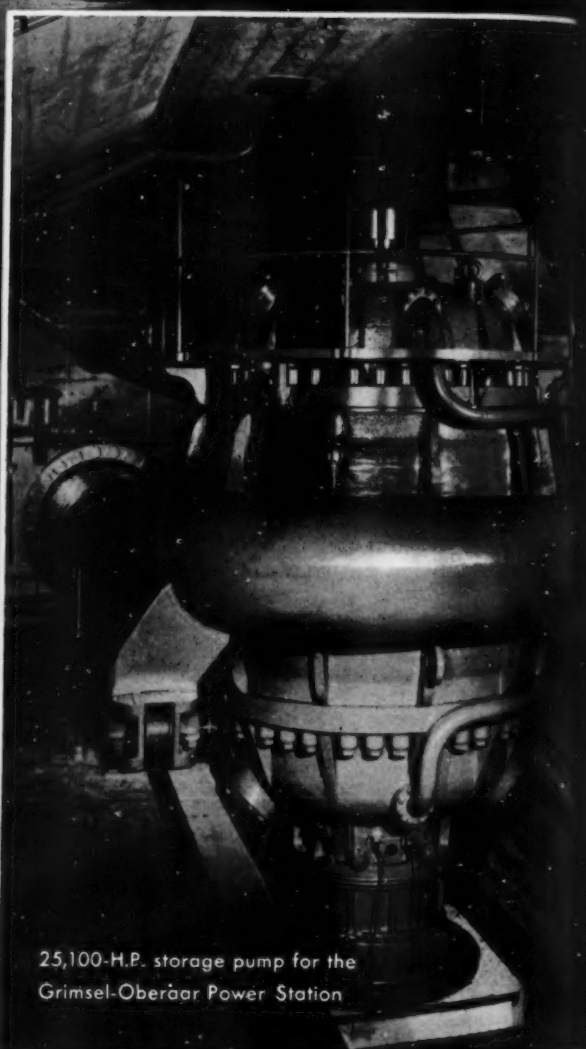
one pump for

Motec Power Station, Switzerland

(discharge 43,050 g. p. m., head 2,060 ft., speed 750 r. p. m., input 30,400 H. P.)

Lake Oberaar, 7,550 ft. above sea level, in the Bernese Oberland, Switzerland



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Abstracts from the World Technical Press

Turkish Scheme Starts Generation

The Sariyar Plant on the Sakarya River, Turkey's largest hydro-electric project, recently picked up its first load—the capital city of Ankara. Two installed generating units, operating under a 250 ft. head, now develop a peak load of 90,000 kW, to be ultimately doubled by the addition of two further units for which facilities are provided. The project, on which work began early in 1952, includes a 354 ft. high gravity-type concrete dam storing 1,600 million acre ft. of water for both power and flood control. A 26 ft. diameter, 3,280 ft. long concrete-lined tunnel delivers the water to vertical turbines in the power house, a 270 by 98 ft. concrete structure, and overlooking the switchyard is an 11 million gallon surge tank, believed to be one of the world's largest single units of its kind. (*Engineering News-Record*, Vol. 158, No. 2, January 10, 1957, p. 60, 2 ff.)

Concrete Semi-Spiral Turbine Casing

Sarda power station, commissioned early in 1956, confronted its designers with many intricate problems, one of which, the turbine concrete scrollcase, is discussed in this article. The concrete semi-spiral scrollcase setting was considered the most appropriate type to suit a static head of about 69 ft., and the stipulations of the suppliers of the plant, The English Electric Co. Ltd., were also in its favour. The design was based on the water-passage dimension fixed by the plant suppliers. The portion at the beginning where the walls of the scrollcase are spanned by a deep upstream main beam, was considered as a frame structure. The rest of the concrete casing was designed as a pipe the cross-sectional area of which varies along the scroll and which is discontinuous on the inner side though it is connected there through the speedring. (Hari Krishna, *Indian Journal of Power and River Valley Development*, Vol. VI, No. 9, September 1956, p. 17, 4 pp., 2 ff.)

Pressure Distribution on Protective Aprons

This study applies specifically to the sloping end of protective aprons set up in conjunction with high-head overflow spillways. Two instances are discussed: (a) an end sill with a "sharp corner," in which the downstream face of the sill slopes 8 : 10 to the horizontal; (b) an end sill, in which the connection between downstream face and horizontal apron is rounded off. The investigation was done by Southwell's method as applied to the case of free-boundary problems, and suitable transformations in curvilinear coordinates for curved boundaries in similar cases, to avoid irregular stars. In the course of the analysis, in accordance with the assumption of high-velocity flow, the contribution due to gravity has been considered separately, and the necessary corrections to the velocities on the free surface and also the resulting contributions to the pressure coefficients for the sills have been given. It is seen that these contributions are relatively small. The computed pressures are compared with the pressures observed in the course of an experimental study of such end sills effected in connection with model tests for Jawai dam. There is good agreement between the results, indicating the usefulness and accuracy of similar design analyses of the action on end sills of protective aprons for high-head overflow spillways. (D. K. Vaid, *Irrigation & Power*, Vol. 23, No. 3, July 1956, p. 389, 13 pp., 11 ff.)

Note: Attention is also drawn to the following papers which appeared in the above mentioned journal: "On the Stress Concentrations for multiple Openings in large Structures," by S. K. Roy, p. 355, 18 pp., 12 ff.; "The Stability of Soil at the Exit End of Hydraulic Structures due to Seepage," by S. R. Datta, p. 373, 3 pp., 1 ff.

Water-Power Development in Pakistan

The following data are included in an article dealing with irrigation and power development in Pakistan. At the time of partition, hydroelectric capacity, restricted to the Malakand plant, was 10,000 kW. Since then 11 schemes have been sanctioned to increase the aggregate capacity to 718,000 kW. Three of these schemes, the Malakand extension, and the Dargai and Rasul projects have been completed, the capacity available being now 62,000 kW. From the multi-purpose projects, civil-engineering works at Warsak (150,000 kW) are nearing completion, and preliminary work is still in progress at Mangla dam (300,000 kW). Following the approval of the project, work is now advancing smoothly at Karnafulli (80,000 kW), while work is held up at the small Gilgit project, owing to financial stringency. (*Indian Journal of Power & River Valley Development*, Vol. VI, No. 8, August 1956, p. 17, 8 pp., 9 ff.)

Priest Rapids Dam

Work has just started on this great development on the Columbia River, which will produce 630,000 kW for Pacific Northwest power users. A remarkable feature of this huge job, involving a record expenditure of $92 millions, is that it has been placed under a single contract. Excavation items total 3,239,000 cu. yards, of which no less than 767,00 cu. yards is in basalt rock for the power house alone; embankment or fill items total 3,352,000 cu. yards. There are no unusual features in the design of the dam, which, except for a light offset on the left embankment, is on a straight line across the river for a total of 9,312 ft. This includes a 1,900 ft. earthfill right embankment, a 260 ft. concrete gravity dam, a 1,142 ft. spillway, a 1,025 ft. power house including assembly bay, and a 4,085 ft. left embankment. The spillway, with an ogee crest at El. 436·5, has 22 openings, each equipped with a 40 by 50 ft. tainter gate putting maximum controlled water level at El. 286·5. It is designed to pass a 1,400,000 cusec flood, the maximum on record being 692,000 cusecs. The power house will have eight 83,000 kVA units, each powered by a 114,000 h.p. Kaplan turbine operating at 78 ft. rated head. Two additional power house bays are being constructed to provide space for future units, should additional upstream storage make it worth while. The site is underlain by massive basalt flows that dip toward the left abutment. This gives good rock foundation under the spillway and power house sections, but, as the basalt dips away at the left abutment, to minimise seepage

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at this point, a 1,000 ft. section of the bank upstream from the dam will be protected by an impervious earthfill blanket. Cofferdams will be built in three stages, the main feature being a high cofferdam around the power house that will permit work uninterrupted from flooding in that area. Work on this high cofferdam is already in progress and should be completed substantially in advance of the 1957 flood season. To date, most construction effort has been spent on accumulating material, a 15 mile access road is being built from Beverly, Wash., and the necessary job site facilities, as well as a bridge across the river just downstream of the dam. To make the job more attractive for tendering, a year was added to the time that was estimated as needed to complete the contract; thus a period of 1,900 days, starting with the actual award of the contract on July 9, 1956, is allowed the contractor to do the job. The contractors, the Merritt-Chapman & Scott Corporation, New York, however, have set their construction schedule at 1,500 days. If this works out, they could receive a completion bonus of $4 million. Design and construction supervision of the work is being handled by Harza Engineering Co., Chicago. (*Engineering News-Record*, Vol. 157, No. 1, January 3, 1956, p. 36, 5 pp., 5 ff.)

Tingambato Project

The first of three generating units at Tingambato, last and most spectacular unit of Mexico's Miguel Aleman hydro-electric system, will soon be on the line. Located 120 miles west and slightly south of Mexico City, Tingambato includes two dams and an underground power house connected by nine miles of tunnels and a 1,127 ft. long pressure shaft. Tunnels, shaft and power house 207 ft. long, 114 ft. high and 39 ft. wide, were blasted out of granite. Ultimately, the power station, the excavation of which took 19 months, will house three 50,000 kW units, thus adding 150,000 kW to the 216,775 kW capacity of the system. The vertical Francis turbines of the plant operate under a pressure of 562 lb. per sq. in. Tingambato constitutes the downstream end of a chain of five power plants with an aggregate head of 6,335 ft. (*Engineering News-Record*, Vol. 158, No. 4, January 24, 1957, pp. 173/174, 4 ff.)

Construction Equipment

Though devoted especially to road-building equipment, this editorial staff report will prove of considerable interest to dam builders, since both these branches of civil engineering use to a very large extent the same kind of construction equipment. Haulage units, shovels and cranes, tractors and loaders, scrapers, drilling and blasting implements, compactors and paving machines, are discussed in separate sections. The report concludes with a chapter on small tools, such as saws, drills, hammers, vibrators and concrete grinding machines, not to mention a pneumatic device that breaks the tyre bead from the steel rim of tractor and scraper wheels. Some of the machines described have a working capacity hardly thought of a few years ago, and are really of outsize class, such as for instance the four big electric rigs at Oahe dam, which carry 13 cu. yard buckets as shovels or 14 cu. yard buckets as draglines, and have actually moved as much as 145,000 cu. yards in two 10 hour shifts; 25 and 30 ton truck cranes; a 24 yard scraper pulled by a 300 h.p. overhung-type tractor with a 218 h.p. engine on the rear to drive the rear wheels. The diesel units powering scrapers are now ranging upwards of 300 h.p. and machines with 600 h.p. engines are planned, while handling capacities of over 40 cu. yards are contemplated. (*Engineering News-Record*, Vol. 158, No. 4, January 24, 1957, appendix printed on buff paper and inserted after p. 104, 32 pp., 58 ff.)

Pleasant Valley: Another Hells Canyon?

It would appear that power projects on the Snake River cannot come to fruition without passing through a protracted stage of violent convulsions. The agitation on the Hells Canyon issue has not died down yet, and the high-dam v. low-dam controversy, complicated by the clash of Federal and private interests, starts afresh around Pleasant Valley project, 24 miles downstream of the discarded high dam at Hells Canyon. For the past year, the Federal Power Commission has had under consideration a licence application from the Pacific Northwest Power Co. for two low dams at Mountain Sheep and Pleasant Valley. It seems, however, that at the present moment a battery of favourable facts is crowding the Administration toward a proponent position on a high dam for the Pleasant Valley site. From the triple aspect of flood control, power potential and irrigation, the Federal high-dam scheme tops opposing plans. On the other hand, the question of costs is still rather confused, for whereas the two low dams advocated by the power combine indicate a total cost of $156 million, generating units included, the Bureau of Reclamation estimates of two years ago for two similar low dams are much higher than this, in fact, about double. Thus, they estimated the cost of the Pleasant Valley low dam alone at $200 million while on the same basis their figure for the Federal high dam now under construction would come to about $275 million. An interesting feature of the six-generator equipment proposed by the power combine for the lower dam is that they would be larger than any at present in operation anywhere else, 49 ft. in diameter with a rated capacity of 144,000 kW and a maximum output of 170,000 kW each. This would leave the 130,000 kW generators at Grand Coulée behind, but would not reach the capacity of the 200,000 kW units now under construction for the Bratsk power station in Siberia. The article concludes with a review of the political aspects of the dispute. There is still a possibility of a partnership which would give the combine the rights to the power from a Federal high dam, or even of the power company building themselves the high dam without any Federal participation. And while the controversies go on, Senator Morse, fresh from his victory over Administration candidate Douglas McKay, intends reintroducing his bill to authorise a Federal high dam at Hells Canyon. (*Engineering News-Record*, Vol. 156, No. 25, December 20, 1956, p. 19, 3 pp., 4ff.)

Fabricated Blondin Runways

This article deals especially with elevated runways built of structural steel as a substitute for ground tracks on dam sites where the latter are ruled out by the lie of the land. Three examples of these fabricated runways are described. At Sambuco, where concreting was affected by means of three parallel 10 ton blondins, the runways of the travelling derricks, 95 m. in

aggregate length, are mounted on a steel structure anchored in the rock of the valley slope and resting, on the dam side, on twin columns of steel sections spaced 7·5 m. and having a maximum height of 16 m. Each runway is submitted to a stress of about 120 tons per metre of length and set of four rollers. At Mauvoisin, the travelling pylons of the three 20 ton blondins move on a runway bedded most of its length on a solid mass of concrete and across two couloirs on steel bridges. At its south extremity, the runway extends far enough above the valley by means of a third bridge. This latter section consists of an assembly of two vertical girders and one horizontal one resting on a triangular bracket anchored in the rock by a horizontal tie-beam. At Moiry, where topographical conditions are more favourable than at Sambuco and Mauvoisin, it was possible to lay about half the length of the runway directly on a concrete floor, the other half, about 70 m. long, being mounted on a steel structure which gradually stands away from the valley slope. The advantage of structural-steel blondin runways is not restricted to costs. Erection can be effected in a very short time, as soon as the components are on site, and, upon completion of the dam, runways and supporting structures can easily be dismantled, and are available for use, if required, at other construction sites. (J. L. Perrenoud and W. Schibler, *Bulletin Technique de la Suisse Romande*, Vol. 82, No. 26, December 22, 1956, p. 457, 5 pp., 10 ff.)

Poirson Thermometric Efficiency Method

We should like to draw attention to the supplement to the above issue of *La Houille Blanche*, which gives a full English translation of an article published on this subject; it is in two instalments, viz., Nos. 4 and 5, Vol. 9, 1954, of that journal, pp. 449-460 and 590-607 respectively.

Melchsee-Frutt Project

This project of the Obwalden Canton is based on the harnessing of the waters of the Melchsee, the average level of which will be raised six metres to El. 1,893 m. by means of a 50,000 cu. m. dam, founded partly on rock, partly on sound moraine, which will create a reservoir 3·7 million cu. m. in utilisable capacity. Since the lake itself drains a catchment of only 7·6 sq. km., a further utilisable storage of 3·8 million cu. m. will be secured by raising the level of the small Tannensee, 82 m. higher up and about 1·5 km. to the west, by means of a 360,000 cu. m. earthfill dam 25 m. in maximum height, 600 m. long at the crest, the two lakes being connected by a tunnel. The upstream section of the power conduit leading from Melchsee to the Hugschwendi power house consists of a tubular steel tunnel 900 mm. diameter with a 7% gradient which connects with the downstream section constituted by a tunnel-housed steep penstock 2 km. in length, with sections 800, 750 and 750 mm. diameter respectively and walls varying in thickness between 7 and 19 mm. A most remarkable feature of this development is that it includes no surge tank. The solution adopted after extensive tests of a tubular tunnel cum penstock, proved not only more economical but also hydro-technically more satisfactory, since the many faults and ravines which cut through the mountain mass might have caused heavy water losses. The penstock reaches the power house over 27 fixed anchorages. The building of the Hugschwendi plant, on the left bank of the stream of the same name, consists of two parts, the machine hall proper, and the switch room, together with the control room, which is adjacent to the machine hall. The generating equipment comprises two Pelton turbines operating under a head of 769 and 819 m. respectively, the discharge being 0·95 and 0·98 cu. m. per sec. respectively. The turbines, supplied together with the penstock and the spherical valve by the firm of Bell & Co., Kriens, each have a capacity of 8,600 and 9,500 h.p. respectively, run at 750 r.p.m., and drive an 8,800 kVA Brown-Boveri generator. The voltage is stepped up from 5,200 to 50,000 V by two outdoor transformers. The installed capacity appears at first sight somewhat inflated. This is justified, however, by the fact that Melchsee-Frutt is a typical accumulation plant, the two storage reservoirs of which enable it to concentrate generation over a few day hours. Production will total 18 million kWh in winter and 19 million kWh in summer, that is to say 37 million kWh yearly. Construction work began in July 1955 and generation is scheduled to start in May. The Tannensee dam, however, is to be commenced only this year so that the development will not be able to operate as an accumulation scheme at full capacity until next year. Investigations are now in progress for the purpose of enlarging the development by pumping into the Melchsee the waters of the Seefeldsee, the highest level of which stands about 65 m. below the Melchsee level. It is also planned to raise the level of Seefeldsee to a utilisable storage capacity of 3·7 million cu. m. There is also the possibility of utilising the 82 m. difference in level between the Melchsee and the Tannensee by building an additional power house between the two lakes. (Dipl.-Ing. Hans Eichenberg, Zurich, *Wasser- & Energiewirtschaft*, Vol. 48, No. 10, October 1956, p. 299, 7 pp., 11 ff.)

Osberghausen Conduit-type Turbine

The power house of the Rheinisch-Westfälisches Elektrizitätswerk at Osberghausen, on the Agger River, which was put into service in July 1956, is equipped with a new type of tubular turbine of Escher-Wyss design, driving a 400 kW asynchronous generator. The main feature of the new design is its speed controller which consists of a planet gearing fitted with a braking device, an instantaneous disconnecting gear being inserted in the clutch between turbine and generator for the purpose of checking the speed of the generator. In short, it constitutes a kind of runaway governor, in which the planet wheels are fitted in the clutch half of the turbine, the inner pinion mounted on the generator shaft, and the outer rim with inner toothing fitted on to the clutch casing. The oil-operated brake, when lifted, disconnects the generator from the turbine every time the generating is stopped whereas it is again applied and thus reconnects generator and turbine as soon as the unit is started anew. There is no danger of the brake getting stuck when kept for a period in the on or off position. The release gear is of a type which has fully proved itself in steam turbines and other machines. The double purpose of the small generating set at Osberghausen is to produce energy and serve as a test unit. A detailed description of the turbine is given and an account of the acceptance tests. (Professor Dr.-Ing. Georg Hutarew, Stuttgart, *Die Wasserwirtschaft*, Vol. 47, No. 6, March 1957, p. 137, 5 pp., 8 ff.)